JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

VOL. 30 NOVEMBER, 1938 No. 11

CONTENTS

All correspondence relating to the publication of papers should be addressed to

Harry E. Jordan, Secretary

→ 22 East 40th St., New York ←

Subscription Price to Members—$7.00

Additional single copies, to members—50 cents

Single copies to non-members—75 cents

JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

Vol. 30 November, 1938 No. 11

DIFFUSING PITS FOR RECHARGING WATER INTO UNDERGROUND FORMATIONS

Chemical Well Cleaning Methods

BY J. HOMER SANFORD

The first public program of returning large quantities of water into subterranean formations originated from a meeting of the New York State Water Power and Control Commission on May 15, 1934. Little did we, at that conference, then realize what widespread interest our experiment in artificially recharging underground structure by mass diffusion would ultimately attract. Long Island's groundwater situation had reached aspects demanding revolutionary treatment. Static levels in Brooklyn and Queens particularly had dropped alarmingly; salt invasion from surrounding ocean waters threatened the Island's capacious fresh water reservoirs. Communities depending upon wells for public supply were greatly alarmed. The Commission faced an emergency beclouded with doubts arising from meagerness of known facts, so the only solution available seemed that of prohibiting any new water wells in Kings, possibly even in Queens County.

A paper presented at the Four States Section meeting, Washington, D. C., October 7, 1938, by J. Homer Sanford, Cons. Engineer-Geologist, 500 Fifth Ave., New York, N. Y.

Uncontrolled, short-sighted drilling, piling up year after year in heavily industrialized areas, had taken no consideration of possible consequences arising from over-concentration of pumping. Salt content in large public supply wells had increased steadily, so in 1932 the New York City Water Department, coöperating with private water service corporations, issued the results of a joint study. This revealed how alarmingly widespread was the depression in static levels under western Long Island. It likewise indicated the steady landward advance of salt water infiltration along the Island's southern shore line. Brooklyn's industrialized area, stretching from Gowanus Canal toward the western end of Newtown Creek, contained a vast cone of depression which is now commonly termed the "crater." Contouring of underground water levels showed a steep pitch eastward from the East River shore line into a trough where groundwater stood about 25 feet below tide. Subsequently, careful determinations have revealed that the bottom of this crater actually stands 40 feet under sea level in certain places. The crater forms a huge irregular crescent whose inner curve along its steepest pitch rims the Brooklyn Navy Yard area. Oddly enough, there exists some sort of sealing phenomenon along the East River's shore that restricts any tendency for salt water to pour rapidly into the crater. The outer fringe of the crater's crude figure is quite irregular, with attenuations reaching in several directions. Since first observed in 1932 this side of the crater has continued to project itself eastward and southward, so at the present time there are few if any spots in Brooklyn where water contained in glacial deposits stands above tide.

COMMISSION'S MANIFOLD DIFFICULTIES

Legislation was hurriedly enacted early in 1933 by amending the Conservation Law to place all large drilling in Long Island under regulation of the Water Power and Control Commission. Drilling had slowed up noticeably that year, in common with general business inactivity, so that only a few applications to drill wells were received by the Commission. This condition changed markedly by the Spring of 1934 due to a 50 per cent water rate increase by the city and the onset of rapid expansion in air-conditioning. These factors combined with an upward trend in business soon brought difficulties into the administration of policies controlling well drilling.

Alternatives confronted the Commission that appeared to be irre-

concilable. On one hand, Long Island's business urgently needed all economies possible from private pumping. On the other, vexatious indications of groundwater depletion and salt contamination demanded caution about permitting any further well drilling. Furthermore, the Commission lacked adequate facilities for investigating violators who sprange up like bootleggers during prohibition. Indeed, the legislature had not even appropriated adequately for proper inspection to make the necessary final approvals of legal installations.

It was at this juncture that the meeting mentioned above was held in Albany. I had discussed with certain quasi-public organizations, including the Brooklyn Chamber of Commerce, the need of coöperation with the Commission. They endorsed a list of suggested policies for presentation to the Commission which were intended to relieve onerous restriction of the industries. Among the ideas proposed to the Commission was one of experimenting with possible methods of diffusing clear waste water back into the ground. Primarily this idea of mass diffusion was proposed as a partial solution to the extreme drop of water levels in the crater. Everyone clearly recognized that only meager evidences existed of any previous examples of successful diffusion. Indeed, no factual data could be found upon which mass diffusion might be claimed a proven venture in even a small way. There were only two scant indications of success in such a program, one being the prevalence of waste water and sewage disposal by private cesspools all over the Island; the other being a common belief among drillers that a well would usually take up water in amounts approximating what can be pumped from it. It may interest you to learn that this latter assumption has proven fallacious, just another shibboleth commonly accepted until efforts are made to investigate the facts. It is true that, when a well screen opens directly into highly permeable waterbearing structure, water will pass back into the ground in direct ratio to the head built up above the static water level. But even though the screen happens to open directly into highly permeable formations the rate of diffusion will drop rapidly whenever sediments are carried into the diffusing well or precipitation of mineral salts occurs in the vicinity of the screen. The shortcomings of ordinary cesspools likewise became evident when attempts were made to increase volumes of water diffused, because an ordinary cesspool has only the limited area of its bottom available for diffusion.

The program faced myriad unknown difficulties, any of which

might easily have wrecked it. Starting from scratch, all knowledge had to be acquired through experimentation, trial and error. Unfortunately, error too frequently predominated. No money was available for testing or research operations. Progress was made solely by inducing applicants who filed petitions to spend a little money in experimenting. Naturally owners did not welcome increased expenditures for water supply development under business conditions then prevailing. Largely their attitude was one of expecting drillers to do whatever seemed necessary to meet requirements in the cheapest way possible. The Commission adopted a policy of granting applications on a five year trial basis provided the petitioner would return all water pumped into the ground. No method was specified for doing this because none was known. Therefore, the proposition rested directly upon the applicant to work out some solution. In the beginning surreptitious or "bootleg" installations were common, and drillers sometimes coöperated in these violations of the law. Ultimately many such illegal wells were traced down and brought under control of the Commission. The general attitude of drilling contractors was, to say the least, noncoöperative. In certain instances they were more than truculent about meeting requirements laid down in the Commission's decisions. Yet one cannot hold too harsh an attitude against them for they feared heavy loss of business through increased sales resistance when two wells instead of one were required. One must remember that business conditions were at low ebb and purchasers favored the cheapest thing offered. As a result, small diameter diffusing wells with perforated casing for screen were favored until they began to clog up. Usually they were 8 in. in diameter but some were as small as 4 inches. These were driven into the first waterbearing formation available in the cheapest way possible, and if they diffused the capacity of the service well by a short test the job was considered complete.

USE OF ILLEGAL CROSS-CONNECTIONS TO SEWERS

Despite the high permeability of surficial sediments in Long Island it was not long before owners ran into trouble from these haphazardly constructed diffusing wells. Whereupon they promptly made pipe line connections to the sewers to dispose of the waste water. Even now it is probable that certain evasions of the law by cross-connections to sewers exist, but in time these will be detected and remedied.

Considering the above facts it seems manifest why error and failure so frequently predominated during the first two years of this modest program for mass return of water into the ground. Persistence in using small diameter ordinary single-cased diffusing wells will probably continue until it is generally realized that they rarely fail to clog. But progress has been made and drilling contractors now appreciate that it is to their own interest to coöperate with the Commission. The fact is now apparent to most, if not all of them, that the drilling business in Brooklyn, and probably Queens, would have been wiped out had the Commission been compelled to prohibit further drilling.

Compared to the backward attitude of certain drillers were noteworthy efforts to learn the truth about diffusion on the part of some. Sweeney and Gray of Long Island City early realized the importance of making diffusion work a permanent thing. They quickly saw that a diffusing pit is not a well working backwards and gave individualized study towards the application of various principles in meeting different factors encountered in different types of formations. They were the first to specialize in digging large diameter pits to form the upper part of a diffusing well and to provide a cylinder of gravel exterior of the diffusion well tailpiece, whereby water could rise freely and be brought into contact with every permeable formation encountered. The Layne-New York Company likewise adapted their large gravel wall type wells to meet requirements and have made several large capacity diffusing well installations capable of diffusing up to approximately 1,000 gallons per minute. It is doubtful, however, whether diffusion as a common practice would have reached its present status but for the numerous installations made by Sweeney and Gray for all classes and types of users. Gradually other drillers are learning the basic facts involved and tending to depart from their earlier advocacy of small diameter diffusers.

Despite its many trials and tribulations, Long Island's groundwater diffusion program has progressed steadily. On January 1, 1938 there were 106 recorded installations diffusing a total of 29 million gallons daily. Ninety-five of these are seasonal operations for air-conditioning. These operate only during the warm weather months so the diffusing wells stand idle about seven months of each year. The remaining 11 installations diffuse throughout the year, handling condensing water in various types of manufacturing plants. These 11 installations handle about 10 m.g.d. The capacity of

diffusing wells for air-conditioning operations ranges from as little as 75 g.p.m. up to approximately 600 g.p.m. The manufacturing plants range from about 300 g.p.m. to about 1,000 g.p.m. There is nothing as yet standardized about diffuser construction which covers wide ranges in types. This naturally would be expected in a period of experimentation such as the program has passed through in its first four years of operation.

PIT AND TAILPIECE BASIC FEATURES OF DIFFUSION WELL

Basically, however, certain features about underground diffusion seem inherent. In the first place, a diffusion well consists of two primary features, the first being the pit, which should be large in diameter; the second, the tailpiece or center pipe through which incoming water is carried to the bottom of the diffusing well. This tailpiece should be constructed throughout of the best quality corrosion resisting material, not less than 8 in. in diameter and preferably as much as 12 in. The tailpiece should terminate with a best quality screen of Everdur or other acid resisting material; having the largest openings practical for the formations encountered. There should be an ample length of this screen and exterior of it there must be a wall of graded filter gravel having a minimum thickness of 2 in. This wall of gravel must continue, without interruption, upward along the outside of the tailpiece and connect directly into the gravel contained in the large diameter pit. In general, the tailpiece must be constructed so that well cleaning operations are facilitated and so that the use of chemical agents, such as dry ice or hydrochloric and sulphuric acids, may be resorted to without damaging the diffuser screen. The casing to which the screen is attached should be heavy genuine wrought iron with suitable heavy couplings to resist rust and other corrosion over a maximum period of time.

Essentially, any diffuser pit should be constructed so as to dispose of the amount of waste water involved, with ample factor of safety against clogging. In the first place, the screen must be adequate in length so that sand pumped from the service well will not quickly block it up. In the second place, the total number of square feet of effective diffusion must be ample because incrustation is inhibited by air which gets into the cooling water during its circuit.

It must be borne in mind that the higher the level, at which diffusion is effected above the formation being pumped by the service well, the better. Obviously, therefore, whenever two water forma-

tions are available it is preferable to pump from the lower one and diffuse into the upper one. But since the problem of temperature rise comes from returning water into the same formation developed by the service well, this discussion will deal with that type of installation.

Russell Suter of the Water Power and Control Commission whimsically describes the contrast between water levels in the diffusion and service wells as a "pimple" compared with a "dimple." In other words, the cone of influence when pumping from the ground is the "dimple" and the piling up of water above the static table when diffusing it is the "pimple." Since many plants in congested districts have limited ground areas, the first problem is that of spacing service and diffusing wells as far apart as possible. In no case should this be less than 100 feet; preferably it should be several hundred feet. Now if an ordinary single-cased well is used to diffuse directly into the formation being pumped, its diffusion is purely a matter of displacement. Consequently, the warm returning water moves directly towards the pumping well because of the draft created by its cone of depression. Naturally the greater the spacing of such wells the smaller is the arc of each circle of influence intercepted by the other. Conversely, the larger is the arc of normal temperature groundwater drafted upon by the pump and the wider is the arc of distribution of warm water away from the cone of depression formed by pumping. Therefore, if there are no other formations for diffusion except into the waterbearing formation, very wide spacing is essential.

There is no means by which re-circulation of warm water can be entirely eliminated if diffused in the above manner. So it is desirable to induce diffusion into all available permeable strata above the main waterbearing structure whether they be waterbearing or dry from standing above the water table. The large diameter pit comes into play for dry strata. The gravel cylinder surrounding the tailpiece affords contact with minor permeable laminae. Since water to diffuse must build up a static head above the normal groundwater table, it is desirable that this static head exercise its influence upon all the permeable strata against which it can be played. When brought in contact with dry though permeable sediments, the water tends to spread downward in cone fashion as soon as these dry sediments are sufficiently wetted. The pit, therefore, should be not less than 30 in. in diameter.

On Long Island it has been found more convenient to use ordinary rolled plate casing about 36 in. in diameter because a man can work with a short shovel and bucket digging this pit casing through the overlying dry strata down to water level. In certain cases where the large diameter casing cannot be withdrawn, holes are burned into it to permit water to pass outward, but it is preferable that the 36-inch casing be withdrawn, at least partially, as it is filled with specially graded gravel. The center pipe or tailpiece is inserted concentrically, it being bailed downward with a bottom shoe that compels gravel to follow around it as the tailpiece progresses. In most cases it will be found that this tailpiece need not penetrate as deeply into the water formation as does the service well. It must, however, penetrate far enough so that when put into service the total capacity to be diffused can be forced into the ground by building up head pressure with the pump.

When the diffusing well goes into operation the first effect is to force the majority of the waste water directly downward into the water formation. Gradually, however, the surrounding upper formations become saturated to set up a sort of "lampwicking" which tends to draw more and more water into them. Once the formations have been saturated it apparently takes many months to dry them out, if the few seasons of Long Island's operation are any criteria. It has been noted that diffusion wells, constructed according to what we now think are proper principles, begin quickly to absorb water in the spring after a winter's layoff.

By diffusing into overlying formations rather than directly at the level drafted upon by the service well, circulation of water back to the pump seems much retarded. Naturally there is a slight tendency for the warmer water to float on top of the normal groundwater table. Therefore, if not forced directly into the cone of influence of the pumping well, it spreads and any tendency to warm up the ground immediately around the pump is minimized.

Of course, in formations where clay or other impermeable sediments overlie the water formation, the use of a large diameter pit in the upper part of the diffusing well may be obviated. However, in any case it seems desirable to have a cylindrical wall of gravel the entire length of the tailpiece, because it seldom happens that ground formations do not contain some streaks or laminations of permeable sediments into which considerable amounts of water may be diffused even though they are not suitable for developing a service well.

It has been found in certain parts of Brooklyn where coarse, highly permeable glacial gravels predominate, that very large volumes of water may be forced into the ground by building up back pressure from the pump. Tests seem to indicate authentically that between 2,000 and 3,000 g.p.m. may be forced into coarse gravels from which it is impossible to pump more than 400 or 500 g.p.m. This opens up intriguing contemplations of what might be done towards returning enormous volumes of water from storm sewers or other flash sources of seasonal runoff, provided the water can be cleared of sediments. During spring freshets vast quantities of water are wasted over New York's Catskill storage dams. Much of this could be utilized for recharging Brooklyn's depleted ground formations at relatively little more expense than constructing great diffusion wells at convenient points.

Recently the Department of Commerce issued Bulletin No. 17 of their Market Research Series covering the effect of city water and sewerage facilities on industrial markets. Results were tabulated from 486 American cities exceeding 20,000 population. These tabulations clearly indicate how widespread has been the effect of excessive water consumption in air-conditioning, not only upon the available public supply, but in overloading the sewerage systems. It has been stated that Chicago's sewerage system has been taxed to such a degree that restrictive measures had to be taken by city legislation. During recent years Chicago has seen a rise in the per capita sewage flow from 75 gallons per day to 200 gallons per day, largely due to the increase in air-conditioning. Diffusion wells may afford relief in many cases of overburdened sewers, for, even though ground formations may not provide adequate water supply for air-conditioning operations, the wastage of water obtained from the city supply can in many cases be passed into the ground. This not only will relieve sewerage overloads but will minimize high sewage dilution which complicates the operation of sewage disposal plants. According to data in the above mentioned bulletin, 353 cities showed a net drop of 4.2 per cent in the water consumption of 1932 over 1931. In 1933 there was a very slight increase over the preceding year. But in 1934 when air-conditioning began to expand, water consumption in 370 of the larger cities jumped nearly 6 per cent. In the last year recorded, 1936, the increase over 1935 was about 9½ per cent. These figures certainly do not arise from normal increase in demand. Nor can they logically be attributed to expansion of industrial con-

sumption. They seem to indicate but one thing, which is that the rapid growth of air-conditioning is approaching a point where demand upon public water supply and consequential overburdening of municipal sewer systems may exceed the capacity of existing facilities to cope with them. The situation is undoubtedly confounding to those city officials responsible for providing adequate public water supply and sewerage, particularly because of its tremendous peak demand in the months of July and August when normal municipal demands are at their highest.

WELL CLEANING METHODS

Turning to the matter of well cleaning methods, it is proper to consider these in their application to diffusing wells since the features are the same as for cleaning ordinary pumping wells. In the order of their importance, well cleaning reagents are dry ice, hydrochloric acid and sulphuric acid. Dry ice comes closest to being universal in its application because it is safest to handle, more readily obtained, and economical. Usually dealers can supply it sawed into cubes approximately 2 inches square, and thus its handling is facilitated. To get the best effects, the top of the well should be so arranged that it can be quickly closed after charging in the dry ice. I have used quantities ranging from 200 pounds to as much as 1,500 lb. at a single charge, depending upon the size and condition of the well. Solidified carbon dioxide diffuses very rapidly under water. I believe it expands to approximately 1,134 times its volume in about ten minutes time. Therefore, high pressure can be built up if the operation of charging and cleaning the well is skillfully conducted. However, there is another feature about dry ice treatment which few well drillers seem to realize. This is the absorption of carbon dioxide in the water, lowering its pH value and making it a solvent of limey elements. This solvent action tends to weaken the incrustants, freeing them up so they can be drawn into the well by ordinary agitation or pumping. Moreover, in waters of high iron content regular charges of dry ice tend to keep the iron in solution and limit its deposition around the screen. Naturally it is undesirable in most cases to lower the pH of water into acid condition. For cleaning purposes, however, this does not matter and, even though the excess of carbon dioxide causes a measurable pH drop for several days after cleaning is finished, there seems no objection to it.

In a diffusing well the outward pressure caused by dry ice expansion forces through the exterior gravel envelope, exerting pressures

many times greater than normally exist. Any diffusing well cleaning method involves several days of thorough pumping to remove sediments and to free up channels of diffusion by reversing the water movement. The dry ice tends to loosen iron incrustants as stated above, largely by attacking intermixed deposits of lime which apparently occur simultaneously with the precipitation of iron salts from the water. Ordinarily, dry ice constitutes the best and most effective means for cleaning a diffusing well, provided adequate pumping and surging follows the dry ice treatment. It seems impossible to develop a service well which will not persistently deliver small amounts of sediments, usually very fine sand, into the diffusing pit. These particles will ultimately become cemented together with iron if cleaning operations are too long delayed. Therefore, certain types of diffusing wells must be cleaned biannually.

When a stronger reagent is needed, it is advisable to combine hydrochloric and sulphuric acids in the well cleaning operation. Most well contractors who use acids seem to prefer hydrochloric acid, but sulphuric has definite loosening effects directly upon the screen itself which the hydrochloric may fail to produce. Acids should never be poured directly into an open well. This is specifically true in the case of sulphuric acid. A $\frac{3}{4}$-inch pipe should be lowered to the bottom of the well topped by a substantial cone or funnel into which the acid can be steadily poured from the carboys. Any acid cleaning program requires several days, not only to permit the acid to do its work but to allow time for intervening test pumping to observe developments. If a well is in bad condition, I suggest that sufficient hydrochloric acid be used to fill the entire length of the screen twice and enough sulphuric to fill it once. The well should first be treated with hydrochloric, filling the screen and allowing it to remain overnight without disturbance. By pouring it carefully to the bottom, the acid being heavier than water tends to fill up inside the screen and slowly diffuse outward through it. The following morning the well should be agitated gently, increasing the agitating operations as evidences of loosened incrustation appear. I find that agitation by what I term "sloshing" is very effective, not only for acid cleaning but for any other well cleaning operation. This consists of passing a tight fitting plunger from the top of the well to the bottom rapidly enough to force water to overflow outside of the casing, provided there is a gravel wall through which it can escape by the plunger's displacement.

Sulphuric acid treatment should follow the first hydrochloric ap-

plication. But in this case only a short period of gentle agitation is safe or necessary. The chemical reaction and boiling caused by the sulphuric acid with resultant expansion of the screen by the heat will usually loosen much of the screen incrustation so that the second hydrochloric treatment on the following day will complete about all that can be done for effective acid well cleaning. The well should be pumped thoroughly between each acid application, particularly following the sulphuric acid treatment. After any acid treatment it is my opinion that a good dosage with dry ice is highly beneficial. The pressure generated by the dry ice seems to knock incrustation loose which the acid did not and thorough surging and pumping after the dry ice dosage usually completes all the cleaning and incrustation removal that can be effected.

It is impossible to lay down any given procedure for cleaning a well because no one can see what is going on and the only way to determine the next step is from study of what happened in the preceding one. Generally, however, I have found that dry ice is the most effective and practical cleaning reagent. In extreme cases dry ice may be assisted by a single hydrochloric acid dosage of from 20 to 50 gallons, simply to dissolve the limey elements in the incrustants.

To sum up cleaning features generally it would appear that acid cleaning is justified only when the screen is known to be in good condition and of material which the acid will not attack rapidly. Only pickling acids should be used. The effect resulting from acid cleaning may not be fully apparent for several months after it has been done. I know of a case at a large New Jersey hospital where the capacity of three wells was increased only about 50 per cent according to tests immediately following the acid cleaning operation. These wells contained very fine Cook strainers but for several months following the cleaning operation the wells continued to increase in yield until they were back practically to their original capacity. Any cleaning operation should provide for thorough pumping between dosages as well as after cleaning has been finished. The use of dry ice is only effective when the top of the well is so arranged that it can be closed to permit building up underground pressure from diffusion of the dry ice. And finally, it is doubtful whether any cleaning operation can be effectively performed unless agitation by means of an adequately weighted plunger is done intermittently to loosen up incrustants as they are freed from the screen or gravel immediately outside of the screen.

ELECTROLYSIS

By George Cunningham

The usual corrosion problem on underground pipes and lead sheath cables has been the control of stray earth current from electric railways or, infrequently, from other multigrounded direct current systems, such as the Edison three wire system.

In recent years, with the gradual abandonment of grounded direct current street railway systems, stray current corrosion of underground plant has been materially reduced and in many cases it has been practically eliminated. At the same time, however, we find the situation involving localized current corrosion has assumed more prominence and has apparently increased in scope due to the fact that much of our plant is now closer to earth potential without the advantages of electrical drainage. It is a well known fact that parts of the underground plant that previously were negative to earth are frequently changed to slightly positive after the trolleys discontinue operation. As those who are familiar with the electrolysis problem know, areas where cable or pipe are electropositive to earth are the danger areas. The degree of hazard is generally indicated by the magnitude of the positive potentials, the period of their duration, and the resistance of the path from the metallic structure to ground.

In a typical case of stray current corrosion, part of the return current of a grounded direct current railway system leaves the rails at outlying points and returns to the rail or negative bus near the point of supply, traveling between these points through the ground water electrolyte and intervening metallic subsurface structures, including cable and water pipe systems. The metallic structures are negative to earth where the stray current is picked up and are positive where the stray current is discharged to earth. Corrosion occurs where the metallic structure is positive with respect to earth and the rate of

---

A paper presented at the Rocky Mountain Section meeting, Casper, Wyoming, September 13, 1938, by George Cunningham, Transmission Eng., Mountain States T. & T. Co., Cheyenne, Wyo.

corrosion is determined by the amount of current per unit area discharged into the earth. One ampere current flowing for a year will cause approximately twenty pounds of iron or seventy-four pounds of lead to go into solution.

In the absence of stray currents, localized current corrosion may occur through the existence of corrosion cells. The potential differences which are responsible for the existence of corrosion cells arise either from some lack of chemical or physical homogeneity of the metal or from some heterogeneity of the environment at the metal surface.

You may be interested in the manner in which we attack an electrolysis problem. As the subsurface structures of many different companies are subject to injury, and the public as a whole has a direct interest in this type of electrical interference due to services dependent upon such structures, the problem is one preëminently adapted to coöperative treatment.

ELECTROLYSIS SURVEYS

In many cities, joint electrolysis committees have been formed, composed of representatives of the several utilities and municipal departments concerned, to investigate the local electrolysis situation and determine by agreement a course of procedure to be followed. These joint committees have proven advantageous in promoting coöperation among the interests concerned in electrolysis questions, in the interchange of information concerning electrolysis matters and specific cases, in developing solutions of specific problems which may be brought before the committee by making concerted investigations and studies, and by recommending solutions of electrolysis problems and extending every reasonable effort to have such recommendations put into effect. Even without the formation of joint committees, informal interchange of information between utilities helps to build up a better knowledge of the areas in which electrolytic damage to underground structures has been experienced.

Generally the first step in the treatment of an electrolysis problem is to make an electrolysis survey to determine the hazard to the underground structures, the extent of damage already done, and the mitigative measures that may best be employed for reducing the danger of future trouble. Where the structures of more than one utility are in the area to be investigated, a joint survey is generally desirable.

A certain amount of physical data covering the plant of the various

interests involved may be necessary. The data are generally in the form of maps showing the location of all subsurface structures such as cables, pipe lines, etc., together with street railway tracks, substations, negative feeders and any existing metallic drainage wires or bonds in use.

There are no reliable methods which can be used universally to determine quantitatively the presence of corrosion damage on underground structures except the method of actual inspection. This is, of course, impractical except for special tests, and therefore other means which give qualitative results are used. The usual electrolysis survey consists in making either indicating or twenty-four hour recording tests of the potential between the underground structure and an electrode of the same metal placed in the ground. Positive potentials generally are considered as indicative of the presence of conditions which may cause corrosion. Potential measurements are of considerable advantage in most cases of stray current electrolysis as the sheath or pipe to earth potentials usually are more than 0.1 volt. Where the potentials are less than this amount, particularly in the absence of nearby street railway systems, potential measurements may have practically no significance. In addition to potential tests, the usual electrolysis survey includes measurements of current flow along the cable sheath or pipe. This is particularly useful in the study of mitigative measures for stray earth currents. Over-all potential drop measurements are also of considerable value.

In the absence of stray direct currents, localized current on the cable shealth or pipe usually is too small to measure without special methods, and may be of little value if known. Measurements of the current discharged into the earth and earth resistivity tests, made by means of an earth current instrument, are of value in special tests. Measurements of over-all rail potential serve as an index of the condition of the railway return circuit.

MITIGATION OF CORROSION

The electrochemical theory of corrosion now is accepted generally as the one which best explains the multitude of facts concerning corrosion. This theory proposes that corrosion is accompanied by a transfer of electricity between positive and negative areas. This entails that there be an electrolyte and an electric potential to maintain the reaction.

Thus two methods are available for the reduction of corrosion on

underground structures. One is to maintain the metal free from moisture and contact with the ground water. The other is to maintain the metal slightly negative to the ground water with which it is in contact. It is doubtful if it would be possible to prevent, absolutely, all corrosion under all circumstances by making the metal negative to the electrolyte. It is believed, however, that if the metal is maintained consistently negative to the surrounding earth, corrosion under ordinary conditions should be so reduced as not to shorten appreciably the life of the structure.

The reduction of stray current leakage from electric railways is an important detail of mitigative measures. If uncontrolled, stray current on cable sheath or pipe may cause very serious electrolytic corrosion damage, but if properly controlled it becomes possible to maintain the underground systems negative to earth and reasonably free from corrosion. The stray current picked up by the cable or pipe systems should be reduced to a minimum by avoidance of incidental or accidental contacts or close proximity of the cable or pipes to other metallic structures from which current might be collected.

CURRENT DRAINAGE SHOULD BE MINIMUM

The stray current is usually drained from the cable systems, and in some cases from the pipe systems, by means of a metallic bond to some point of lower potential on the railway negative return system, but preferably not to the rails. The current drained from an underground system should be the minimum which will maintain the entire system negative to earth. Insulated joints are used extensively in electrolysis mitigation for interrupting current flow and isolating sections of a system from stray currents. If there are other cable or pipe systems in the vicinity, care must be taken not to cause positive conditions on these plants. The best engineering solution in such a case may be a coöperative drainage scheme. Competitive drainage by several utilities usually results in poor economies and ineffective drainage.

Measures which tend to reduce the amount of stray current from grounded rails are increased track leakage resistance through the use of rock ballast, the reduction of the potential gradient or voltage drop per unit of length of rail through the use of improved bonding at rail joints, and the use of additional negative feeders or substations.

Subsurface structures which cannot be maintained negative to earth by picking up stray currents are subject to possible corrosion by localized currents. In order to protect them it may be necessary to resort to preservative coatings of organic materials such as laquers, enamels, complex structures of such materials as pitches or asphalts with jute felt or paper, etc. All coatings must be impervious to ground water. Pipe coatings of various types have failed in the past, but considerable progress has been made within the past ten years in developing adequate preservative coatings.

Cathodic protection, or forced drainage, may be applied effectively to underground pipe and cables under certain conditions. This method has complications in its use in certain parts of cities where pipe may be close to the underground plants of other utilities. In such a case a joint installation is sometimes desirable.

FORCED DRAINAGE

In all types of forced drainage the purpose is to prevent corrosion of underground structures by adequately reducing positive potentials of the underground structure with respect to its environment, so that appreciable current will not be discharged from the drained structures into the earth at any locations. This is usually accomplished by means of an external direct current voltage connected between the underground structure to be protected and a special ground. The negative side of the external source of potential is connected to the underground structure. In this way the potential of the structure with respect to the surrounding earth is lowered, the amount depending upon the value of the drainage current. Forced drainage may also be used as a booster to increase the current in a long drainage wire to a negative bus.

Ground systems for forced drainage installations must have a substantial metallic core around which is a special environment used to prevent disintegration or corrosion of the core. Galvanized pipe not smaller than $1\frac{1}{4}$ inches and in lengths of 20 feet or old steel rails where available in sections of not over 10 feet usually provide the most practicable core. For the environment fine carbon in the form of breeze coke gives satisfactory results. Experience in several locations has indicated that approximately one-half ton of breeze coke should surround each 10-foot section. The direct current source may be local batteries, generators, or rectifiers installed at

convenient intervals along the subsurface structures. This means is used to protect the underground gas main distributing system of New Orleans with excellent results.

In application to our cable plant under favorable conditions the extent of the spread along the cable of the counteractive effect of negative current application to the sheath is remarkable. As an example, in actual practice a current of 0.75 amperes was found in a specific case to maintain about eight miles of cable at least 0.1 volt negative to earth whereas the previous potential ranged from 0.01 to 0.20 volt positive to earth.

In special cases where it is desirable to correct small positive potentials localized in a small area and where only a small amount of drainage current is required, the use of zinc plates or slugs buried in the ground and connected to the subsurface structures by small copper wires has proven effective in maintaining the structure negative to ground for considerable periods.

In conclusion I wish to stress again the importance of coöperation between the various interests involved in an electrolysis problem, since adequate handling of the problem can be undertaken only when all interested parties are willing to coöperate.

PROGRESS REPORT
AMERICAN RESEARCH COMMITTEE ON GROUNDING

At the spring meeting of the American Research Committee on Grounding, the Technical Subcommittee submitted a progress report to the representatives of the 14 national organizations which are coöperating in this movement. The report covered six additional field investigations and several sets of laboratory tests.

In one of the six cases investigated no trouble was reported, the investigation being made because in the construction of a large building the electrical isolation of all protective grounds from the piping systems had been attempted. Electrical tests and inspection showed, however, that such isolation had not been attained.

In the second case reported on, the complaint was of leaky water pipe joints, and the trouble was apparently due to a combination of high water pressure and poor workmanship in the original installation. In another case, where the complaint was of a failure of boiler tubes, steady readings of small direct current were obtained at the boiler, indicating galvanic action. On the pipes and grounding conductor close to the service entrance the readings were small and fluctuating, indicating stray electric railway currents.

In the three remaining cases the complaints were of discoloration, or other impairment of the quality of the water delivered through taps on customers' premises. Of these three cases one showed no electric current present on the pipes, another was cleared up by lime treatment of the water, and the third case was remedied by reducing the temperature of the hot water to 140°.

To date the Technical Subcommittee has investigated some 26 field cases where complaints have been made, and where it was suspected that stray alternating or direct current might have been a factor in producing the trouble. In none of the cases so far investigated, however, has the trouble been found to be due to alternating

---

Submitted by C. F. Meyerherm, Albert F. Ganz, Inc., 511 Fifth Ave., New York, N. Y., and Secretary of the American Research Committee on Grounding. Chairman of the Committee is H. S. Warren, 420 Lexington Ave., New York, N. Y.

current on the water piping system, although, on the other hand, investigations have not shown that such current may not have contributed to the cause of the complaint in some of the cases. A number of significant factors have been found, including the tendency of the water in question to corrode the piping or other metal, the effect of dissimilar metals in the piping system creating galvanic couples, and corrosion of hot water supply boilers due to abnormally high temperatures. Further field investigations are to be made as cases are reported to the Committee by water companies where troubles such as corrosion, impairment of water, sparking, or electric shock are suspected as being caused by electrical grounding.

Field work is being supplemented by laboratory investigations of the fundamental electro-chemical principles involved. It is believed that these will throw considerable light on the causes of the water companies' difficulties.

Laboratory tests on samples of metals simulating pipes have been made. These tests have indicated many factors that enter the problem of the effects of superimposed alternating current on galvanic cell current and further tests to determine the fundamentals involved are now being undertaken.

SHOULD PLUMBING INSTALLATION BE APPROVED BY THE WATER DEPARTMENT

By E. F. Dugger

For the past two decades water works engineers and management have applied their knowledge and devoted their energy toward improvement in the quality of both domestic and commercial water supplies. The result has been that the average water works plant in this country is furnishing to its supply system, water that is free from taste and odor, and in most cases sterile, with practically zero color and turbidity. Standards for a domestic and commercial water supply have, during the past two decades, risen to such a high point that nothing of an inferior quality will be accepted by the patrons.

With these high standards for the effluent of our filtration and sterilization plants we are still unable to guarantee to the customer a pure and wholesome supply of water. This is due mainly to the fact that little or no consideration has been given to the installation of water piping and plumbing fixtures in the consumer's home. Water works departments have thoughtlessly maintained that all liability as far as they are concerned ceases at the meter.

It is proper that we as a group give careful and systematic thought to the elimination of some of the many hazards to health that exist in the average building supplied with water. The effect on the customer's health is the same whether the water supply becomes polluted at its source through improper purification or sterilization or becomes unsafe due to entrance of pollution into the supply lines in the building. In other words it is our obligation to supply pure and wholesome water to our consumers, and to do this it is necessary that the entire system, from the source to the spigot, be of proper design, and under definite supervision and control.

There are numerous cities that exercise absolutely no control over

---

A paper presented at the Virginia Section meeting, Lexington, Va., August 26, 1938, by E. F. Dugger, Gen. Mgr., Newport News Water Works Com., Newport News, Va., and former President, A. W. W. A.

the installation of water pipes in their community. Under such conditions it is obvious that there is something drastically wrong—when large expenditures are being made in purification plants and pumping stations, and so little thought is being given to the many sources of pollution that exist on the customer's premises.

A study of the report issued by the United States Public Health Service, covering the summary of inspection of plumbing in Federal Buildings in New York and Detroit, will disclose the necessity of correcting the plumbing hazards in the average home.

In New York City for example there were 4,702 flushometer type closets inspected, and 4,702 of them were disapproved. While this is the highest percentage of defects found in any of the fixtures inspected, yet the total percentage of defective or disapproved fixtures is exceedingly high. There was a total of 21,928 plumbing fixtures inspected in New York City; 14,844, or approximately 67 per cent, were found defective or disapproved. It is reasonable therefore to expect that if the large percentage of defects was found in government buildings where unusually rigid inspection and carefully prepared specifications for installation are maintained, we can expect an even greater percentage in the average home.

U. S. PUBLIC HEALTH SERVICE REPORT

I am quoting herewith an excerpt from this report of the U. S. Public Health Service, which I believe will be of interest to you.

"In a study of several of the buildings inspected in New York, subsequent to the completion of the survey, circumstances conducive to the spread of contamination, generally reduced water pressure or creation of partial vacuum in water lines, were examined for the purpose of comparing their frequency of occurrence. In one building constructed recently, those factors necessary for the development of a disapproved fixture into a potential health hazard were almost nonexistent. Thus, while several of the fixtures in this building were listed as disapproved, the degree of hazard to health in their use was exceedingly small. In another building studied, conditions potentially contributory to the spread of contamination were observed to occur quite frequently. While about the same percentage of fixtures was listed as disapproved, in the older building as in the newer building, the health hazard surrounding their use was many times greater. It is evident, therefore, that fixtures which might be permitted in one building, if allowed to exist in another building, where different physi-

cal forces come into play, might contribute measurably to the health hazard in the latter building.

"In the two cities in which surveys were initiated, the work as planned was successfully completed. In 1935 it was possible to find a sufficient number of young engineers seeking employment who could be trained and utilized on the projects. Whether they would be available in times of normal employment conditions is doubtful. These surveys demonstrated that other required personnel could not be secured from the Works Progress Administration relief rolls in adequate numbers and, further, that employment restrictions necessarily imposed by Works Progress Administration rules for the utilization of work relief labor were such as not to permit the establishment of the most effective organization in an undertaking of this character.

"That cross-connections and defective plumbing fixtures or installations exist in federal buildings cannot be denied. Some defects were of immediate danger to the health of the water users in the buildings in which they were located; others were of remote significance unless other variables were brought into play. As defects were found in the New York buildings in about the same numbers as in Detroit buildings, it can be concluded that the same conditions exist in federal buildings in all other cities. Also, it seems reasonable to conclude that similar conditions could be found in other public buildings generally."

Many of us seem to forget that water can flow both ways in any supply or distribution line. For example, at any time the pressure is low on the street mains during peak period, and the cook in the kitchen is attempting to get water from her spigot, part of the water will come in driblets from the city main, and the balance siphons back from the toilet tank on the second floor. Conditions such as these must have our attention.

Greater demands upon our supply and distribution system are being felt each year, mainly through the demand in the business and higher class areas for air-conditioning. This demand all comes in the summer season when dry weather is to be expected, and is most likely to necessitate the use of auxiliary supplies which may be far from satisfactory from a health viewpoint. Further, this increased demand for water may over-tax existing treatment plants and lead to a reduction in the efficiency of the treatment given unless immediate steps be taken to increase elevated storage or increase main sizes. We know that, with low pressures, increased contamination is bound to follow.

No permit for a direct connection from your water supply to a cooling system unit should be permitted. All water used for a cooling system should be discharged into an open tank so that any possibility of contamination by the introduction of toxic refrigerants or gases to city mains will be positively eliminated. Water supply inlets to collecting pans in air washers, de-humidifiers, and to cooling towers must always be above the highest possible water level. The re-use of condensing and cooling water in the domestic supply system should also be prohibited, and above all see that the installation of any air-conditioning system and equipment is permitted only by skilled and licensed men who understand the health hazard, and also have a full knowledge of the action of back siphonage.

I am not nearly so much interested in having the water department approve plumbing installation, as I am in having the plumbing hazards that exist in all our systems eliminated. I am interested in finding out what is the best method to accomplish this, and when it is found, getting to work to eliminate these potential health liabilities.

ULTIMATE AUTHORITY IS LOCAL HEALTH DEPARTMENT

I have felt for a number of years that, due to our existing State Law, the ultimate authority for the correction of these conditions is and must be placed in the local health department. There are, however, few of the small cities or towns, that could afford the full time service in the health department of a qualified sanitary engineer, and I assume that all are agreed that, for satisfactory results looking toward the correction of these problems and the passing upon plans and specifications for new installation, the services of an experienced and qualified sanitary engineer are necessary.

Assuming that we all agree on these two points, first, that the authority for the correction of existing health hazards lies in the health department, and second, that the qualified person to handle the inspection of supervision must be a qualified sanitary engineer, it seems logical that his remuneration would of necessity be derived from at least three departments: first, the health department; second, the public works department, where plans and specifications for buildings must of necessity be approved; and third, the water department where all information concerning the system must be secured. A proper man well equipped will have no trouble in working successfully under these three departments.

Discussion by C. E. Moore.* I believe that the question of plumbing ordinances and codes should be established by and under the administration of a plumbing inspector, who may also be the building inspector.

I feel that the manager of the water department should not be burdened with any of the problems of the administration and enforcement of the plumbing code. I believe he should have the right of inspection, but only the duty so far as it pertains to his own interest as distinguished from the responsibility.

I believe the plumbing code should provide for and the plumbing inspector should be responsible, that all water connections to pieces of property should have their own cut-off beyond the water department meter and should be prohibited operating the department cut-off.

Both departments should have a rule prohibiting cross-connections with some other source of supply, except in a fashion eliminating any possibility of cross-feeding.

The plumbing code itself should provide that any modern type of seat should be constructed, so far as its water feed is concerned, so as positively to render it impossible, upon shutting down of the water supply line at any point on or off the premises, for any water in the seat trap to be syphoned out and backed into the water supply line. The water department cannot be the guarantor of absolute continuity of pressure in the water system; neither must it be burdened with responsibilities for any damages resulting from such failure.

The water department is concerned with safe delivery of pure water to the customer. The customer's condition of being in a position to receive service under all varying conditions and without harm to the water department and its other customers is more properly a function of the building and plumbing inspector's department, this department working in coöperation with the health department.

*Presented by D. R. Taylor for C. E. Moore, Vice-pres. and Treas., Roanoke Water Works Co., Roanoke, Va.

EXPERIENCES WITH CAST-IRON, STEEL AND REINFORCED CONCRETE WATER MAINS

By Humphrey Beckett

In discussing cast-iron, steel and reinforced concrete water mains there may be very little, if anything, new on the subject that will be of interest to those engaged on this type of work. The general methods of installation have changed but little in recent years, with the exception of new types of mechanical joints and the use of sulphur base compounds.

The manufacture of cast-iron pipe has undergone radical changes with the advent of the centrifugal casting method, and the fabrication of steel pipe has been greatly facilitated by the perfecting of the automatic electric welding machines and the ability of the rolling mills to put out large plates, reducing joints to a minimum. Spirally welded pipe can be made with smooth inside wall. The absence of lapped plates and rivets in the steel pipe and the smooth walls of the cast and spun pipe make easy the application of a spun lining either of cement mortar or of a bituminous enamel material in cast-iron pipe and in steel pipe of small diameter.

In the larger diameter steel lines the lack of rigidity makes it inadvisable to use the thin cement mortar lining as provided in Federal Specifications W. W. P.—421 for cast-iron pipe, and the use of a thicker lining reinforced with wire mesh spot welded to the pipe makes the cost excessive. The diameter at which the thin lining should be omitted depends, of course, upon the wall thickness of the pipe. While most cities seem to be adopting the use of linings for trunk line mains, the movement toward the general use in small diameter distributing mains seems rather slow. The city of Washington, while lining all of its trunk lines in recent years, is now planning to clean and line some of its important old ones, but, as yet, has not committed itself to the use of linings for service mains. This is

---

A paper presented at the Four States Section meeting, Washington, D. C., October 7, 1938, by Humphrey Beckett, Engineer, Water Dept., Washington, D. C.

due to the fact that a city of this size has, at all times, an enormous amount of pipe and fittings in storage and the effect of the weather on both cement mortar and bituminous enamel linings, when exposed for several seasons, is problematical.

It is interesting to note, however, that in 1932 we purchased several thousand feet of 30-inch cast-iron pipe lined with cement mortar, in accordance with Federal Specifications W. W. P.—421, and owing to the fact that a new street was dedicated after the purchase of the pipe the proposed line was shortened by 500 feet. This unused balance was placed in open storage and in 1935 some of it was installed. In making a closure it was necessary to cut about four feet from a length of pipe. This was done on the job site with flogging chisels and the workmen were instructed to proceed in the usual manner, taking no precautions to save the lining. Although the lining was crazed, the cut was made with no loosening of the lining and no spalling back in excess of one-half inch. There are several lengths of this pipe still in the open, and, while the lining shows crazing, there appear to be no loose sections and I would not hesitate to install it, if needed—this after an exposure of six years.

There seems to be quite a divergence of opinion as to the best type of pipe to use. The three types under discussion have their advantages and disadvantages, but any one of them could be designed to hold any desired pressure and to last for centuries if costs were not considered. Therefore, it seems to me that the problem of the engineer is to design for needed pressure, desired length of life and long maintenance of carrying capacity, and award the contract to the lowest responsible bidder. The question of pressure, of course, is no problem except as to joints. The question of life of cast-iron is no problem if building for 100 years as is the practice in this city. The question of life of steel and reinforced concrete pipe is to a certain extent theoretical but based on established facts. Knowing the absolute protection given steel by concrete I believe we underestimate the life of reinforced concrete pipe. A few years ago a connection was made with a 48-inch reinforced concrete main that had been in service for six years and the steel cylinder and reinforcing hoops were as clean as the day they came from the mill. The life of steel pipe, I believe, can be safely computed from the data obtained by soil corrosion studies.

The question of long maintenance of carrying capacity is more of a problem. Reinforced concrete pipe, owing to the fact that there

can be no tuberculation, seems to have the advantage in this respect. When the connection with the six year old line previously mentioned was made, the interior was carefully examined and with the exception of a few very small voids, probably caused by air bubbles when cast, was in perfect condition. The useful life of thin cement mortar and bituminous enamel linings for ferrous metal pipes is unpredictable as there are not sufficient data at hand on which to base an assumption, but from their present performance they should hold up well. It must be borne in mind, also, that re-conditioning methods have recently been developed by which large diameter mains can be cleaned and lined at a very reasonable cost.

The discussion to this point has dealt with pipe sections alone without taking joints into consideration. Joints for underground water mains are of three general types: lead joints, sulphur base compound joints, and the mechanical rubber gasket type. The life of the lead joint is indisputable. Unfortunately, they are unreliable under vibration and when settlement occurs under heavy pressure, unbalancing the joint spacing, the lead creeps out on the released side. Internal lead joints, where the pressure tends to hold in the lead, are not vulnerable under this last condition. However, under ordinary settlement with average pressures a leaded joint line is flexible, which is a most desirable feature in cast-iron lines. If leaded joints are to be used in high pressure lines it is advisable to use the double grooved bell and harden the lead by the addition of about 4 per cent tin and 1 per cent antimony.

SULPHUR BASE COMPOUNDS WITHSTAND VIBRATION

Sulphur base compounds, while appearing to lack flexibility, give remarkable performance under vibration and settlement owing to the healing of leaks when the joint is disturbed. Seven years ago, 250 feet of 12-inch main, with sulphur base compound joints, were lowered three feet at the maximum point without shutting down the main, although the pressure was 75 pounds per square inch. Owing to the short length of line exposed, the main locked, but as it was on a curve at this point the trench was widened and the main allowed to go out and down at the same time. The joints were four years old and the lowering was accomplished without leakage at that time, or since. There is no way of determining the life of materials of this type, but they have been in use for approximately forty years.

The use of rubber packed mechanical joints in Washington is

comparatively new, the first having been installed in 1933. They give a remarkably tight line and most of our trunk lines above 20 inches in diameter laid since 1933 have been installed with this type of joint. To date, not a joint has failed. The metal coupling should be designed to equal the life of the pipe, and while the life of the rubber is unpredictable it is a well known fact that one of the best methods known for preserving rubber is to keep it moist and exclude light. This type of joint gives splendid results on bridges as it takes care of expansion and contraction (if pipe is properly spaced) and seems to be immune to damage from vibration. The gasket will probably deteriorate faster than if underground but will undoubtedly give long service.

Methods of installation are generally the same throughout the country, but there seems to be some disagreement as to the use of blocking under the pipe. Personally, I do not believe that any permanent advantage is gained by its use. If the blocking goes down, the joint is disturbed; if it holds, the pipe between blocking points is left acting as a beam. Blocking does facilitate lining up and jointing the pipe. Undoubtedly, the best method is to scoop out a cradle to one-fourth or one-third of the diameter of the pipe to give greater bearing on the undisturbed trench bottom. In small diameter mains this would all have to be hand work and the excess excavation for bell holes would further increase the cost to such an extent that considering the little trouble with mains installed by the usual method it can hardly be justified. Large diameter mains with internal joints requiring no bell holes can undoubtedly be laid at less cost by this method.

CAREFUL WORKMANSHIP NECESSARY TO GOOD JOINTS

In making joints the utmost care should be used as, in my opinion, leaking joints, unless caused by abnormal conditions, are the result of careless workmanship. In leaded joints, failures under normal conditions are almost entirely due to improper yarning, a honeycombed pour, or to making two pours, the second after the first has cooled. Sulphur base compound failures are due to moisture, grease or dirt in the joint, material burned or poured at improper temperature. All of these can be avoided by the exercise of a little care. The proper temperature of sulphur base compounds can easily be determined by observing the surface of the material in the melting pot, which has a mirror like appearance when right. Joints are

often badly poured due to the lowering of the temperature of the material between the melting pot and the joint. The workman making the pour should carefully observe the material in the pouring pot before pouring. Leaking mechanical joints are rare and are caused by slack bolts, tightening up unevenly around the joint, or by taking too much deflection in the pipe.

In cutting into existing mains reinforced concrete is a different problem from that presented by metal pipes. The Water Division of Washington recently connected three 48-inch cast-iron mains with a 78-inch reinforced concrete main having no steel cylinder. The three cast-iron mains were below the concrete main, crossing at different angles. All mains were first exposed, angles, elevations and circumferences carefully measured, and a detailed drawing made showing the method of connection. As no section of the concrete main was to be removed from the line, the plan called for a split sleeve to clear the 93-inch outside diameter of the concrete pipe by about one-half inch all around, with a 73-inch outlet in one of the halves. The edges were beveled for welding, one seam to be on top and the other on the bottom. Two end rings, also split, were provided in the shape of a mechanical joint fitting the pipe, but loose enough to be revolved when assembly was made. These end ring sections were to be welded to the sleeve on the inside diameter of the sleeve, joints matching joints. Two split follower rings were also provided to hold the rubber gasket in position. The two halves were then to be assembled around the concrete pipe and held together by means of eight sets of bolting lugs, four at the top and four at the bottom. The assembly was then to be revolved sufficiently to bring the bottom seam up to avoid underneath welding. After this seam was welded the assembly was then revolved back into permanent position and the top seam welded. The follower rings were then to be placed around the pipe and welded. The gaskets, cut diagonally, were then to be placed in position and follower rings bolted up. Grout, one part cement and two sand, was then to be pumped into the space between the sleeve and the concrete pipe through two boiler flanges tapped for 2-inch pipe at the bottom of the sleeve until it overflowed at two similar openings on top. Bids were solicited for labor and material, including three cone valves, 4-way manifold and all necessary fittings to connect with the three cast-iron mains. Award was made to the lowest bidder and the whole work was completed as

planned with no trouble, resulting in an absolutely tight job. All of the work was supported by a concrete mat.

DESIGNED OWN PIPE CUTTER

Cutting into cast-iron mains by hand cutting methods is a very simple proposition, but is always attended by the hazards of cracked pipe, or a break back to such an extent as to cause a defective joint. The Washington Department has a cutter designed and built in its own shops to cut 48-inch cast-iron pipe. It consists of a split sleeve bolted around the pipe with sufficient clearance to allow it to be centered by means of set screws. The tool carrier is also a split sleeve with gear teeth on the perimeter. This fits around the stationary sleeve with an overhang of about three inches, with a V-shaped tongue riding in a V-shaped groove in the stationary sleeve. On this stationary sleeve is bolted a gear train which meshes with teeth on the revolving tool carrier. The tool carrier is fitted with four tool holding slots and the best results have been obtained by using only two tools, the front or leading one having a V-point and the follower a square point. The tools are fed by automatic star feed. The operation is effected by a portable motor getting its current from a portable generator set. This cutter turns out a regular shop job and will cut 48-inch class "B" pipe in about an hour. The objectionable feature is the time it takes to set up and center the stationary sleeve. As the set up for the first cut can be made prior to shutting down and draining, the only lost time is for the second set up, and considering the advantages gained it is not a serious drawback.

ANOTHER CUTTER OF FOREIGN MAKE

We also have a cutting machine of foreign make for cast-iron pipe. I believe that in general principles it is ideal for pipe cutting, but it should be of greater power and more rugged construction. The cutting is done by a circular saw which might be classed as a thin milling tool. The entire assembly consists of an electric motor of about $1\frac{1}{2}$ horsepower and a machine encasement all in one unit, and a chain made up of links of varying lengths supported at each joint or hinge by a roller.

The chain is fastened to the machinery and motor unit by a coiled spring at one end and a turnbuckle at the other. The saw is carried on the saw spindle which protrudes from one side of the machinery

encasement. The motor and machinery unit is supported on the pipe by four wheels, two front and two rear. The front, or transmission, wheels have sharp teeth and the rear ones are the smooth double edge type.

The operation of the saw is effected by the enclosed motor, which turns the blade and moves the carriage automatically. By adjusting the tension of the chain the apparatus is pressed on the pipe, causing the transport wheels to press their slight indentations into the surface of the pipe. The saw is raised by means of a snail controlled by a handle on top of the carriage. This machine will cut pipe from 8-inch diameter up. In cutting the smaller pipe only the shorter chain bars are used. The saws provided with the machine had a slight set but not sufficient to prevent binding and the blade would heat, overload the motor or break, and finally wreck the clutch. This was repaired and a set of saws furnished with side cutting teeth which are a great improvement over the original blades.

For cutting steel pipe we have a small apparatus designed to hold an acetylene torch. It consists of a light carriage supported on four sharp toothed wheels held on the pipe by a light roller chain. It is revolved by hand by means of a worm drive. The torch can be set at an angle and will give a good clean beveled edge if welding is desired. Owing to the even speed that can be secured and the fact that the torch is held steadily at an equal distance from the pipe much less gas is consumed.

A comparison of the costs of maintenance of the three types of pipe cannot be made from experience in Washington as our steel and reinforced concrete pipe have been in service only a few years and, to date, there have been no maintenance costs. The same can be said of our recently installed cast-iron mains.

Nearly all of our maintenance troubles are caused by old leaded joints and old horizontally cast small diameter iron pipe having a very thin wall on one side. Breaks or ruptured pipes on large diameter cast-iron mains are not of frequent occurrence in this city and what trouble we do have is generally caused by improper construction, such as not allowing sufficient clearance over other structures or allowing a pipe to rest at one point on a rock, or insufficient blocking or strapping of fittings.

I believe that all of the three types of pipe, if properly designed and properly installed, will give many years of useful service and will cause but little worry to those responsible for maintaining an uninterrupted supply of water to large cities.

LEAKAGE FROM THE DISTRIBUTION SYSTEM

By Arthur M. Field

It is, perhaps, elementary to say that the problem of maintaining a water distribution system that is at least 75 per cent tight is of primary importance to a city whose water supply must be pumped, because of the direct cost of pumping, or where the existing source of supply is limited and the water must be conserved. It is important also to cities obtaining their water from an adequate gravity supply because, although there may be no direct pumping cost and no immediate possibility of consuming the entire capacity of the source, the available water for distribution is limited by the size of the supply mains, even if there is an abundance of water at the source. When the capacity of these supply mains is reached, a new capital investment will be required for enlarging or adding an additional supply main, or perhaps building new storage reservoirs. When, as in most cases, the supply main is of any considerable length, the debt charges for a new line may easily be greater than direct pumping costs.

Assuming that the water department of the city is managed in a businesslike manner, with sound plans projected for future development, it is of great importance to know the volume of water that must be put into the mains to take care of the probable domestic and commercial needs five, ten and fifteen years hence. Unless some standard percentage of the water sold to the water supplied can be set up as an objective to maintain, no very sound planning for the future can be undertaken. This means then that some definite program of inspecting for and checking leakage should be carried out.

There is another reason for systematic checking of leakage in water mains that is probably not of great importance to water officials, but is important to the city, and that is the damaging effect on the street foundation when the ground is saturated by leaking water mains. Street departments spend thousands of dollars to drain streets and to prevent ground water or surface water from getting

A paper presented at the Virginia Section meeting, Lexington, Va., August 25, 1938, by A. M. Field, City Mgr., Winchester, Va.

under the pavements and blowing them up. Yet it is possible for a broken main or service lateral to discharge thousands of gallons of water a day into the subsoil under a street, saturate this soil for a considerable distance, and then work its way into a sewer without ever showing up on the street surface.

In one of the surveys made in Winchester for water leakage we discovered a single leak on a service lateral wasting 50,000 gallons of water per day. Water from this leak had saturated an area of 50 square yards and then entered the sanitary sewer. We don't know how long it had been running, but it must have been for some time. This 50 square yards of street paving had entirely lost its original contour and later had to be dug up and rebuilt.

The City of Winchester derives its water supply from three limestone springs. As is characteristic of springs in limestone country, the supply is variable, ranging from a maximum of $6\frac{1}{2}$ million gallons daily in very wet seasons down to $1\frac{3}{4}$ million gallons in a drought season, and probably averaging $3\frac{1}{2}$ million gallons a day. The minimum flow is approximately our present average daily consumption, and our peak daily consumption this summer has run up to $2\frac{1}{2}$ million gallons a day.

We pump through our distribution system against two reservoirs with a combined capacity of $3\frac{1}{4}$ million gallons. We are building a belt system of large mains as rapidly as current revenues will permit, but at present in periods of heavy draft certain sections of the city suffer loss of pressure, one of our mills having complained frequently this summer of this trouble. We need an additional or new supply, completion of our belt system of large mains, and additional storage capacity in the opposite end of the city from the present reservoirs. Under these conditions, until our supply can be increased, it is of the utmost importance to conserve our water and to maintain as tight a system as possible.

In 1916 the city was supplied by only one spring and at that time we were pumping about one million gallons of water per day, which was dangerously close to the dry weather capacity of that spring. We employed the Pitometer Company to make a survey for leakage and we made a close follow up, repaired the leaks, and reduced our pumpage from 1 million gallons to 670,000 gallons per day. I emphasize the close follow up with the repair gang because in 1928 the city faced another water shortage and made a survey, but apparently did not follow up the repair of the leaks, because very little reduction was made in pumpage.

Again in 1935, not because a shortage threatened, but in the interest of economy, we had a complete survey made. As a result, our pumpage, which had run approximately 615 million gallons in 1934, dropped to 532 million gallons in 1935, and we sold 56 million gallons more water than in 1934, making a net reduction in the amount of leakage and fire use from 244 million gallons to 104 million gallons and increasing the percentage of water accounted for from 60 to 80 per cent.

That great vigilance is required on a system is indicated by the fact that in 1936 and 1937, the first and second years after our last survey, our percentage of water accounted for has decreased from the 80 per cent in 1935 to 73 per cent in 1936 and 71 per cent in 1937. At the time of making our last Pitometer survey we purchased Pitometer recording equipment and installed in manholes permanent taps for inserting the Pitot tubes. Men in our water department were trained in the use of the equipment so that we could periodically check the system ourselves.

Due to the fact that we pump through our distribution system to our reservoirs, we cannot easily determine our real flow of water entering the system. We find that the simplest method is to cut off our pumps and to measure the rate of flow with a portable recorder, from the reservoir, through the entire night. This is done monthly.

Knowing what our minimum night rate was at the completion of our survey with a system 80 per cent or more tight, and if we find that our minimum rate of flow, which generally occurs about 2 or 3 o'clock in the morning, is considerably above what the standard should be, and unless there is some readily explainable reason for the increase, we know that we must make a more aggressive search for leakage.

We also have our distribution system divided into zones which can be closed off and the recorder set up on one main feeding this zone. Thus, in the event of a sudden large increase in flow, it is possible to determine very soon the zone in which the leakage is occurring and to concentrate our efforts within that zone. A quick survey is made to determine any unusual use and if the increase cannot be thus accounted for, we then have to take to the listening devices to determine where the leak may be. This work must be done largely at night when other noises interfere least.

Because of the difficulty of locating leaks with any great degree of accuracy with the geophone or aquaphone, last year we designed

and constructed a portable electric amplifier operated from storage batteries. With this instrument it is possible to set up once or twice in a block on a hydrant or valve and determine the sound of a leak within that block and then to trace it down to almost exactly above the point of leakage.

Our experience is that most of our leakage comes from a considerable number of relatively small leaks, leaks of two to five thousand gallons a day. The tremendous increase in the use of heavy trucks, we believe, is responsible for loosening joints in the mains, which should be laid deeper in the ground for the traffic of today. The other chief cause of leakage is the deterioration of old galvanized services which are being replaced gradually with copper. All our new services are of copper.

We have been able to make a larger number of repairs than we might otherwise, by replacing services by digging a hole at the main and another at the meter behind the curb and then using a pipe pusher in reverse, pulling the new copper service in attached to the galvanized service as it is pulled out, thereby eliminating considerable excavating and saving the street surface.

When we find a block in which several service leaks occur and where the houses have been constructed at about the same time, we endeavor to replace all of the services in that block with copper as fast as possible. As I stated, most of our leaks are relatively small and accordingly we are making a practice of conducting a rather continuous survey with our amplifier to locate and check leaks.

Our surveys for comparisons of "accounted for" water with what we have set up as a standard have discovered, in addition to leaks, illegal use of water through fire bypasses or other means and in one case we found that a meter was making one complete revolution between readings, so that the consumer who had been paying for 250 thousand gallons of water was actually using 1 million and 250 thousand gallons of water each quarter.

Aside from conserving our water supply, we have been able, by reason of reducing our leakage, and also by the installation of more efficient pumping equipment which was decided upon as a part of our survey, to reduce our pumping costs from approximately $8,200 spent in 1934 for electric current to approximately $6,000 in 1937, and during this same period our sales of water have increased by approximately $5,000.

LAYING THE 30-INCH SUBAQUEOUS MAIN UNDER CURTIS CREEK, BALTIMORE

By J. Milton Kinnear

At the southeast corner of Baltimore lies Marley Neck. Within its borders are the U. S. Coast Guard Depot, the U. S. Quarantine Station, several large industrial plants, and a considerable area of idle land. Bounded on three sides by navigable waterways, divided by the main highway to settlements along the Chesapeake Bay, and lying adjacent to important manufacturing districts, it offers inducements for both industrial and residential developments. At various times in the past, attempts have been made to develop tracts of land along Pennington Avenue and Marley Neck Road, but for several reasons, including the lack of a reliable water supply, the developers' plans have never passed the paper stage.

When in 1933 improved pavement was proposed in Pennington Avenue and Marley Neck Road, the Baltimore Bureau of Water Supply decided to encourage the development of Marley Neck by extending its distribution system into the territory. Accordingly, in 1934, the 30-inch main in Pennington Avenue was extended to the west bank of Curtis Creek and a 12-inch main laid from the east bank along the highway to the city line. Later, in 1937, construction was begun on the joining of the two dead ends by a 30-inch main under Curtis Creek.

Curtis Creek at the point of crossing is about a quarter of a mile wide. Spanning it required 1,200 feet of flexible joint pipe and about 150 feet of standard bell and spigot pipe. The main for about half its length runs parallel to and 175 feet from the center line of Pennington Avenue Bridge, curving at the ends to tie in with the previously constructed piping. At an elevation of five feet above mean low tide, special fittings are used to change from metropolitan to bell and spigot pipe. These fittings are also designed to compensate for any

A paper presented at the Four States Section meeting, Washington, D. C., October 7, 1938, by J. Milton Kinnear, Asst. Civil Eng., Bureau of Water Supply, Baltimore, Md.

future settling of the main. The 30-inch valves at each end of the job are insulated, with rubber gaskets and fibre sleeves and washers, against stray electrical currents. The main is laid in strata of sand and clay with a cover of about four feet below the river bed, except where it dips below a proposed channel deepening of 13 feet. No piling was used to support the pipe as tests made before designing indicated a bottom firm enough to bear the load safely. The underwater main is covered to a depth of two feet with a special backfill and over this a 2-foot layer of river muck.

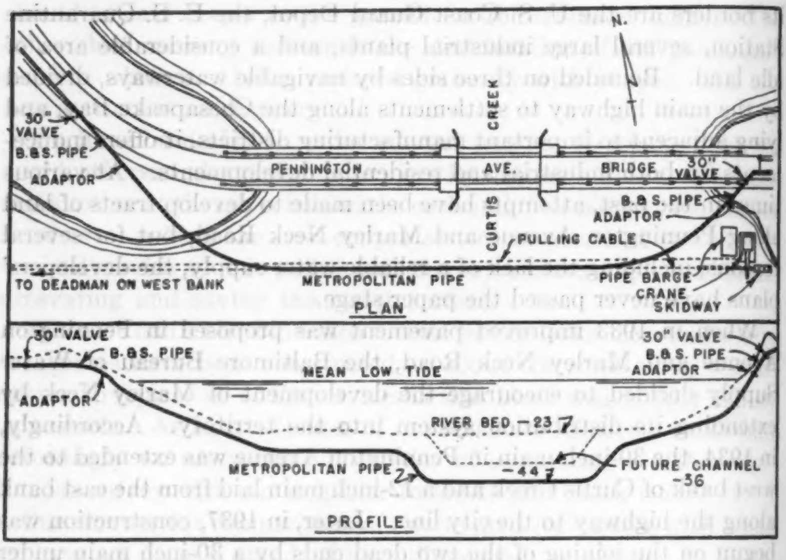


FIG. 1. PLAN AND PROFILE

Construction of sub-aqueous water main, Baltimore

The contractor's original plan was to build the main on a curving cradle attached to a barge, which would lay the main in place as construction progressed and the barge, with the cradle, was moved ahead. Due to the many necessary changes in length and curvature of the cradle resulting from the uneven profile, he abandoned the idea and decided to construct the main on a skidway near the east shore and pull it, as it was built, towards the opposite shore by means of a cable running from the front end of the piping, through an anchored pulley on the west bank, and back to the power plant located at the skidway.

The flexible joint pipe used on the job is of the metropolitan type. The metropolitan joint is a ball and socket affair, and a poured lead joint is used to hold the spigot in the bell. The spigot and the cast-on ring on the inside of the bell are carefully machined to a spherical surface. The ring serves a three-fold purpose: first, it centers the spigot in the bell, thus assuring a uniform lead space; second, it acts as a lead stop which requires no packing; and, third, it provides a hard bearing surface for the spherical spigot to ride on as the joint is flexed. Lead grooves on the inside of the bell hold the lead stationary with respect to the bell, and this facilitates caulking.

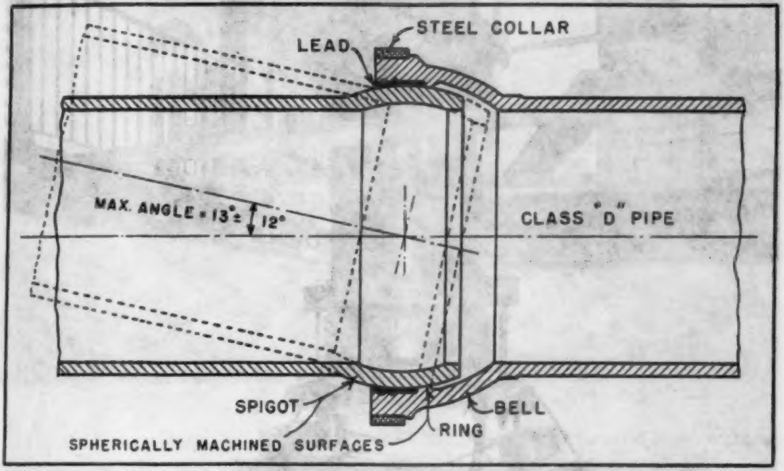


FIG. 2. METROPOLITAN JOINT
A ball and socket design with poured lead joint

The dashed lines in fig. 2 show the joint approaching its maximum deflection. The lead is just beginning to be compressed and, upon further deflection of the pipe, will flow out of the joint. At its maximum deflection of approximately 13 degrees, the barrel of the pipe bears against the inside edge of the bell face and threatens to crack it. To prevent fracture, a 1 in. x 4 in. steel collar is shrunk around a machined surface on the outside of the bell. The collar is also useful in protecting the pipe during handling and shipping.

There being no available data on the pulling capacity of the metropolitan joint, the Bureau of Water Supply decided to test it in a manner which would be similar to actual conditions experienced in

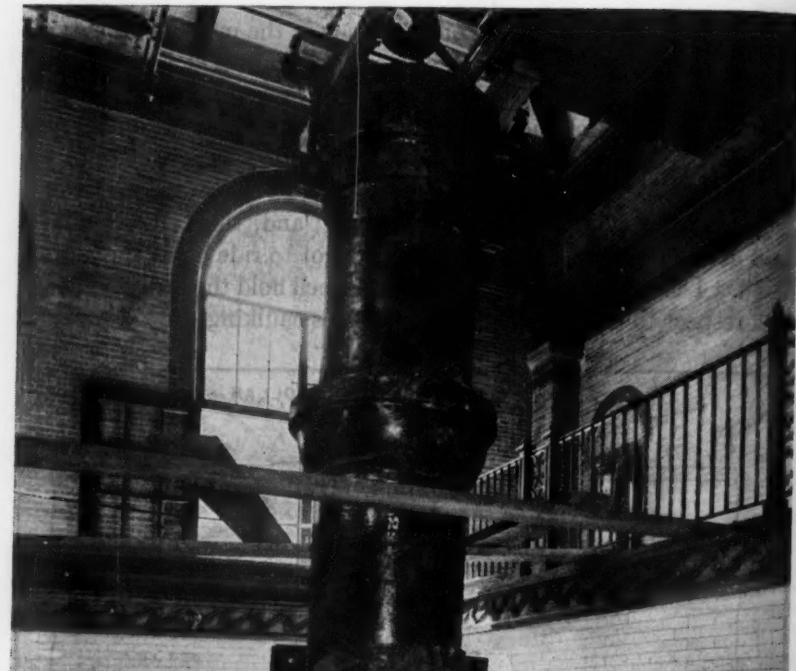


FIG. 3. ARRANGEMENT FOR TESTING JOINT STRENGTH

Raising and lowering the load with an overhead crane duplicated the pulls that would be put upon the joint in construction.

following the contractor's proposed method of construction. Two special fittings were laid in a horizontal position, the joint was poured and bulkheads were set in each open end. The resultant tank was subjected to hydrostatic pressure. The joint failed at 170 pounds per square inch when the spigot pulled out of the bell, forcing the lead ahead of it. The total maximum force applied to the joint was about 80 tons.

Another test was made with the same fittings in a vertical position; the upper end attached to the hook of a crane and 15 tons of pig lead attached to the lower end (15 tons was the crane's capacity and also

FIG. 4. LAUNCHING WAYS

The first three lengths of the main are shown; they are poured and caulked and ready to slide into the water until all but the last bell is submerged.

the contractor's estimate of the maximum force required to pull the main across the creek). The whole assembly was raised and lowered 100 times to represent the repeated pulls on the joints during construction. There being no movement of the lead during the test, 15 tons was considered a safe pulling force, and the contractor was instructed to use a safety link in his pulling chain which would not permit the load on the joints to exceed that amount.

Dredging having been completed, the contractor was ready to build the main. The photograph shows the set-up at the launching ways. The skidway consists of parallel rails, spaced about two feet apart, and resting on timbers supported by pile bents. Beyond the Whirley Crane is the pipe barge. The pipe was transferred from

here to the deck of the crane where a protective covering of tallow and white lead, applied by the manufacturer, was removed from the machined surfaces of the spigot and cast-on ring. A graphite grease lubricant was then applied to these surfaces. Later, when the joint was poured, the grease would burn out upon contact with hot lead and the graphite would remain as a thin film between the lead and the spigot, thereby permitting easy flexing of the joint.

The first three lengths are shown on the skidway. The joints have been poured and caulked and a bulkhead placed on the front

FIG. 5. USE OF STEEL DRUMS
Eight fifty-gallon drums were tied to each joint

end. A snubbing rope, which prevents the pipe from moving down the ways, later on was released and the pipe permitted to slide into the water until all but the last bell was submerged. Other lengths were then laid on the skid and the procedure repeated. After about eight lengths had been laid, the pulling cable was attached at several points along the main in such a manner that the pulling force was distributed equally between these points. This was done to reduce the individual load on each of the first few joints where the greatest stress occurred. The stress in each of the following joints was re-

duced by the frictional resistance, along the river bed, of the pipe between each joint, and the front end of the main. Beginning with the first length behind the pulling cable, and prior to each pulling operation, eight (8) 50-gallon steel drums were tied to each joint. These helped make the empty main semi-bouyant so that it would drag lightly along the river bottom.

As the depth of water along the pipe line varied from zero to about 50 feet, and as the drums would crush under 35 feet of water, four of them were attached to the main with long ropes and four with short ropes, the latter carrying the load from the skidway, through the shallow water to about the 25-foot depth, where they would be released by the diver, and the load thus transferred to the longer roped tanks which were just beginning to submerge.

After passing under the channel the main began to rise toward the opposite shore. As the water became shallow the drums on their long ropes lost their effectiveness. In order to make them take up the load again, they were drawn down by the Whirley Crane to the pipe line, and there tied with short ropes by the diver.

Little difficulty was experienced in pulling the first 25 lengths, but river water entering through the joints, which would leak as soon as they were flexed, then made the the main too heavy to be pulled by the specified maximum force of 15 tons. To empty it, the main was sealed with another bulkhead at the back end and charged with 50 pounds of air pressure. This forced the water out through a valve in the front bulkhead and through a hose to the surface of the river where progress could be noted. When all the water was exhausted, pulling was continued and construction resumed, but at an increasingly slower rate for now the front end of the main was moving across the level stretch under the channel.

As each joint dragged along the river bottom and down the slope toward the bottom of the channel, the bell face, acting as a scoop, would carry clay to the bottom of the slope and deposit it there as pulling operations lifted the joint off the bottom. The result was an accumulation of sticky clay in which the main would sink and refuse to be pulled through. At one time there was about 5 feet of muck lying on top of the pipe for about 100 feet of its length.

MUCK BOTTOM CAUSES BREAK IN LAYING

In addition to blowing out water, it was now necessary, before pulling could be resumed, to send out the crane to lift the main above

the muck in which it had settled. This in turn caused more trouble, for, one day after the water had been blown out and the main was under 50 pounds air pressure, the crane was lifting the structure above the muck when the thirty-fifth joint, which probably had been weakened by too much flexing, suddenly kinked. The reaction from the 50 pounds pressure on the suddenly formed bend was too great for the joint. It parted completely. As later investigation showed, it had bent beyond the allowable deflection of approximately 13 degrees and the lead had been squeezed out.

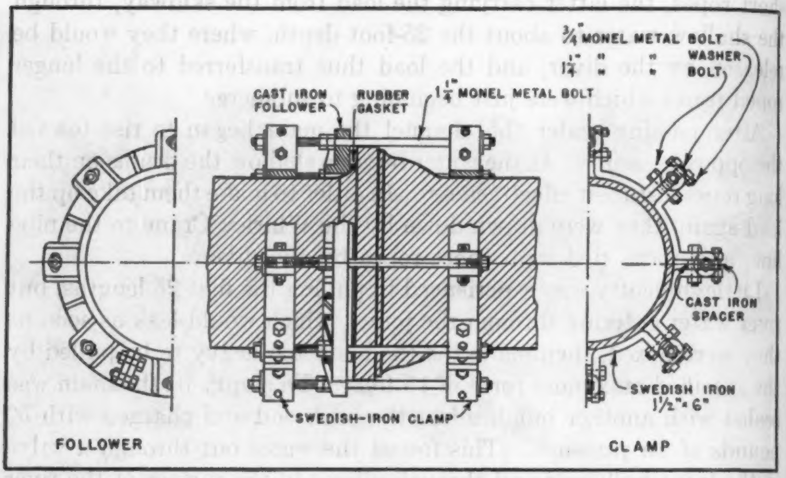


Fig. 6. Clamp and Follower

These were specially designed as the solution for making the final joint.

After the break the first 35 lengths were pulled ahead into their final position and a bulkhead and pulling cable were attached to the front end of the second section. The idea was to pull the second section ahead as it was constructed and make an underwater joint between the sections. But, when pulling was resumed, the thirty-eighth joint parted. It had probably been weakened when the other joint blew out. The three unattached lengths were lifted to the surface and returned to the skidway to be relaid. The bulkhead and pulling cable were then attached to the thirty-ninth length and construction continued without further trouble.

After the final joint was poured on the launching ways, the second section was pulled ahead until its front end was a little past the back

end of the first section. Both sections were allowed to fill with water and to settle into position. The opening between the two was closed by springing the pipe into place.

The joint was packed with lead wool and caulked. As the lead wool would not stand up against flexing or any pulling stress, the joint was held together and stiffened with a clamp especially designed for the purpose. The collars of the clamp are built up with 1 in. x 6 in. Swedish iron sections, roughly U-shaped and assembled with $1\frac{1}{4}$- and $\frac{3}{4}$-inch monel metal bolts. Each section is separated from the adjacent section by cast-iron collars, thus permitting room between the flanges for the $1\frac{1}{2}$-inch monel metal bolts that serve as stiffeners. To further insure water-tightness a cast-iron follower ring forces a rubber gasket against the lead space. Pressure on the follower is maintained by the adjusting and lock nuts on the $1\frac{1}{2}$-inch stiffener bolts.

When the three unattached lengths were brought to the surface they contained considerable river muck. As the open ends of the pipe remaining on the bottom evidently had also permitted the entrance of muck into the main it was considered necessary to clean it. Cleaning was done by forcing a go-devil through the line with water pressure from the city water system. Passage of the scraper through the one-quarter mile of piping required about 10 minutes, but flushing continued until the water flowing from the main was clear.

After cleaning, the tie-in was made with the mains in Pennington Avenue. Kinks in the underwater joints, which were located by soundings with the aid of a diver, were straightened out. The diver also inspected each joint to determine if lead had been forced out of any of them. The worst condition showed about $\frac{3}{16}$-in. of lead squeezed out beyond the bell face. This was not bad and was easily recaulked. The contractor then charged the main with 50 pounds of air pressure to locate leaks by escaping air bubbles. Divers recaulked the joints until bubbling ceased.

Before placing the special backfill, the main was subjected to the several tests called for in the specifications. First was the hydrostatic test, which required that a pressure of 175 lb. per sq. in. be maintained in the pipe for 10 minutes. The contractor pumped the pressure up to 177 lb. and within the required 10 minutes dropped only 3 lb. without any additional pumping. Water was then blown out and the air pressure brought up to 50 lb. per sq. in. This test

required a very calm day to locate air bubbles rising to the surface. We were fortunate with the weather and were able to locate and caulk several small leaks.

The final test was for water leakage to be measured with a water meter. The main was charged and the valves at each end were closed. Any further flow of water into the main passed from the distribution system through a meter and small piping installed for the test. Our standard specifications allow a leakage of 50 gallons per inch of diameter of pipe per mile of length per 24 hours. For this particular job, this indicated an allowable loss of 375 gallons in 24 hours. Results of the test showed an actual loss of only 21 gallons, or less than 3 gallons per inch of diameter per mile, in the 24 hour period, which is less than 6 per cent of the allowable.

Naturally we were very much pleased with the results of the tests and therefore permitted the contractor to go ahead with the final stage of the contract—backfilling. To protect the main against the corrosive effect of the highly acid river water, it was covered to a depth of two feet with a special backfill consisting of a mixture of sand, powdered clay, limestone dust, and limestone chips. Chips were not specified, but were used after experiments showed that the other three ingredients would not give a mix stiff enough to mound over the pipe. Mounding was an economic necessity as the trench in places was as wide as 40 feet, with side slopes rising one in four. The purpose of the special mix was not to prevent river water from reaching the piping, but to neutralize the acid which is highly corrosive. No doubt, water pressure and the cementing qualities of limestone will (in time), make the mass waterproof.

Mixing of the special backfill was done as carefully as though it were a good grade of concrete. A mixing plant was set up consisting of a store house, a road paver, and a hopper. After the measured ingredients were placed in the mixer, sufficient water was added to make the mix plastic enough to flow through the tremie, which was used in placing. (We found that a dry mix would choke the tremie pipe.) From the paver the mix was dumped into a hopper and then into a bottom dump bucket resting on a truck.

The bucket was hauled out on the bridge where a crane lowered it to a barge. From the barge it was picked up by the Whirley Crane and conveyed to the tremie which was set up on a pile driver. The pile driver hookup was very convenient for it could raise or lower the tremie as the profile demanded.

Placing of the special backfill went very smoothly and upon its completion it was covered with a 2-foot layer of river muck. This was placed very gently so as not to disturb the lower layer. Its purpose is to protect the special mix against erosion, but it also has the added virtue of being a poor conductor of electricity, thus reducing chances of electrolysis. Except for cleaning up and repairing any surface structures which had been damaged, this completed the job.

PRACTICAL APPLICATION OF THE LANGELIER METHOD

By Charles P. Hoover

It is the purpose of this discussion to point out the practical applications of the Langelier formula for determining pH saturation rather than to attempt an explanation of its derivation. W. F. Langelier, Associate Professor of Sanitary Engineering, University of California, presented the formula in a paper at the convention of the A. W. W. A. in June, 1936, under the title "The Analytical Control of Anti-Corrosion Water Treatment." It was published in the October, 1936 issue of the Journal. By use of the formula, the pH saturation of water can be computed from the results of a water analysis, i.e., from the alkalinity, the calcium, the dissolved solids and the temperature, the pH saturation can be calculated mathematically.

Water is saturated with calcium carbonate when no additional calcium carbonate can be dissolved in it. The solubility of calcium carbonate in water, however, depends upon the carbon dioxide content and the pH value of the water. For instance, water with 60 p.p.m. of carbon dioxide and a pH value of 7.0 will hold 500 p.p.m. of calcium carbonate in solution, i.e. as calcium bicarbonate; whereas, water with 1 p.p.m. of carbon dioxide and a pH of 8.3, will hold in solution only 100 p.p.m. With water having a pH up around 9.4, only about 13 or 14 p.p.m. will be held in solution. Practically speaking, water that is saturated or supersaturated with calcium carbonate will deposit scale on surfaces with which it comes in contact, but water that is not saturated will not lay down scale. Too much scale in a distribution system is objectionable, but a little deposited as a protective coating prevents red water trouble and, it is believed, retards corrosion.

Whether or not water is saturated with calcium carbonate can be determined either by the so-called marble test (precipitated cal-

---

A paper presented at the Eighteenth Annual Ohio Conference on Water Purification, September 27, 1938, at Columbus, Ohio, by Charles P. Hoover, Chemist-in-Charge, Water Softening Plant, Columbus, Ohio.

cium carbonate chemical balance test) or by the Langelier formula. The modified precipitated calcium carbonate chemical balance test outlined below can be made in a few minutes, and we believe the results are sufficiently accurate for all practical purposes. The test is made as follows:

1. Determine either the alkalinity or pH value of a sample of water.
2. To another portion of the same water, add chemically pure washed precipitated calcium carbonate in excess (a teaspoonful to 150 ml.). Stir a few minutes, and allow to settle, and filter. Determine alkalinity or pH as in 1.
   (a) If the alkalinity or the pH value in 2 is greater than in 1, the water is not saturated with calcium carbonate.
   (b) If the alkalinity or the pH value in 1 and 2 are equal, the water is in chemical balance with respect to calcium carbonate.
   (c) If the alkalinity or the pH value in 2 is less than in 1, the water is supersaturated with calcium carbonate.

The Langelier formula is based on mass law equations and other principles of physical chemistry, and although the derivation of the formula is extremely technical and difficult to understand, its practical application is easy. In order to use the formula conveniently in our own laboratory, Merrill L. Riehl, of our staff, prepared a nomographic chart, which is included with this paper. Suppose the results of analysis of a sample of water show, in parts per million: total solids—300, calcium—40, and alkalinity—60; at a temperature of 25°C. By referring to the nomograph, a total solids-temperature graph will be found. For the particular water in question proceed as follows: Find the point where the 300 total solid vertical line crosses the 25°C. temperature curve. A horizontal line from this point projected across the vertical line marked "column 1" will, at the point of intersection with column 1, indicate the temperature-total solid constant. In this particular instance, the figure will be 2.26. Align this point with the given value of calcium, i.e. 40, on column 3 by laying a ruler from the point at 2.26 (column 1) to 40 on column 3. Now locate the point where the ruler crosses column 2 or pivot line; then align this point with the given value for alkalinity, i.e. 60, on column 5, and the ruler will cross column 4 at the pH saturation, which, in this particular case, is approximately 8.2.

Mr. Langelier has not only contributed a new procedure for deter-

mining the pH saturation of water, but in addition has introduced a new term in water works terminology, i.e., *the saturation index.* What is the saturation index? Answer: The saturation index is the difference between the actual pH and pH saturation (in respect to calcium carbonate). For example, if the actual pH is 7.6 and the pH saturation is 8.1, then the saturation index is −0.5, and the water may be expected to be corrosive. If, on the other hand, the actual pH is 8.4 and the pH saturation is 7.8, then the saturation index is +0.6 and the water may be expected to lay down scale.

In a recent article on corrosion control (August, 1938 Journal of the A. W. W. A., p. 1357), George D. Norcom makes the following comments regarding the Langelier formula:

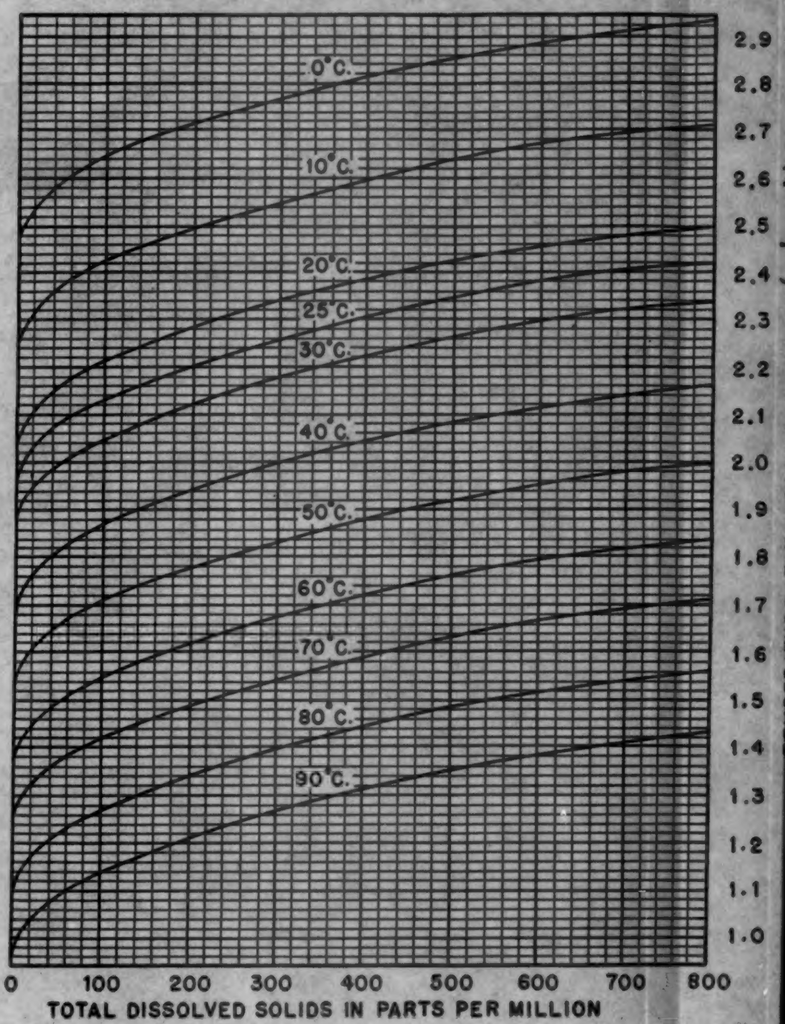
"Unfortunately, this formula requires a complete chemical analysis of a water in order to provide the necessary data for its solution, and in many cases such a complete analysis is lacking. Moreover, if anything is added to the water, such as lime or other alkali, a change in chemical constituents is brought about, and another complete analysis is required. Apparently every change in treatment requires a new analysis. It would appear that this method is too cumbersome for ordinary control of chemical corrosion treatment. This formula does not take into account the oxygen, or organic content of the water, does not predict the treatment necessary, and, therefore, should be considered only in connection with the formation or lack of formation of calcium carbonate deposits."

Mr. Norcom concludes that the modified marble test as outlined above is superior for practical use.

Others have said, "Well, you have the formula and the index, so what?" In fairness to Mr. Langelier, it should be explained that he did not intend (according to my own interpretation of his paper) to convey the idea that by maintaining a positive index, all corrosion difficulties in water distribution systems would be overcome. He did, however, recognize that the maintenance of a positive index is a factor in controlling corrosion, and has provided a new scientific method for determining it. It has one great advantage over other methods, it seems to me, in that the saturation indices can be calculated on water supplies over the years already passed, that is, of course, provided alkalinity, calcium, total solids and temperature records have been kept. A comparison of the known behavior of these supplies in the light of their saturation indices may, it seems to me, be of very great value in the study of this important subject

GRAPH AND NOMOGRAM F

COLUMN 1



GRAPH AND NOMOGRAM FOR DETERMINATION OF pH SATURATION BY LANGELIER'S FORMULA (APPLICABLE WITHIN pH RANGE 7.0 - 9.5)

DATA REQUIRED FOR DETERMINING pH SATURATION.

(a) TOTAL ALKALINITY, AS PARTS PER MILLION OF $CaCO_3$

(b) CALCIUM IN PARTS PER MILLION.

(c) TOTAL DISSOLVED SOLIDS, IN PARTS PER MILLION.

(d) TEMPERATURE, IN DEGREES CENTIGRADE, AT WHICH pH SATUR

INSTRUCTIONS FOR USING CHART

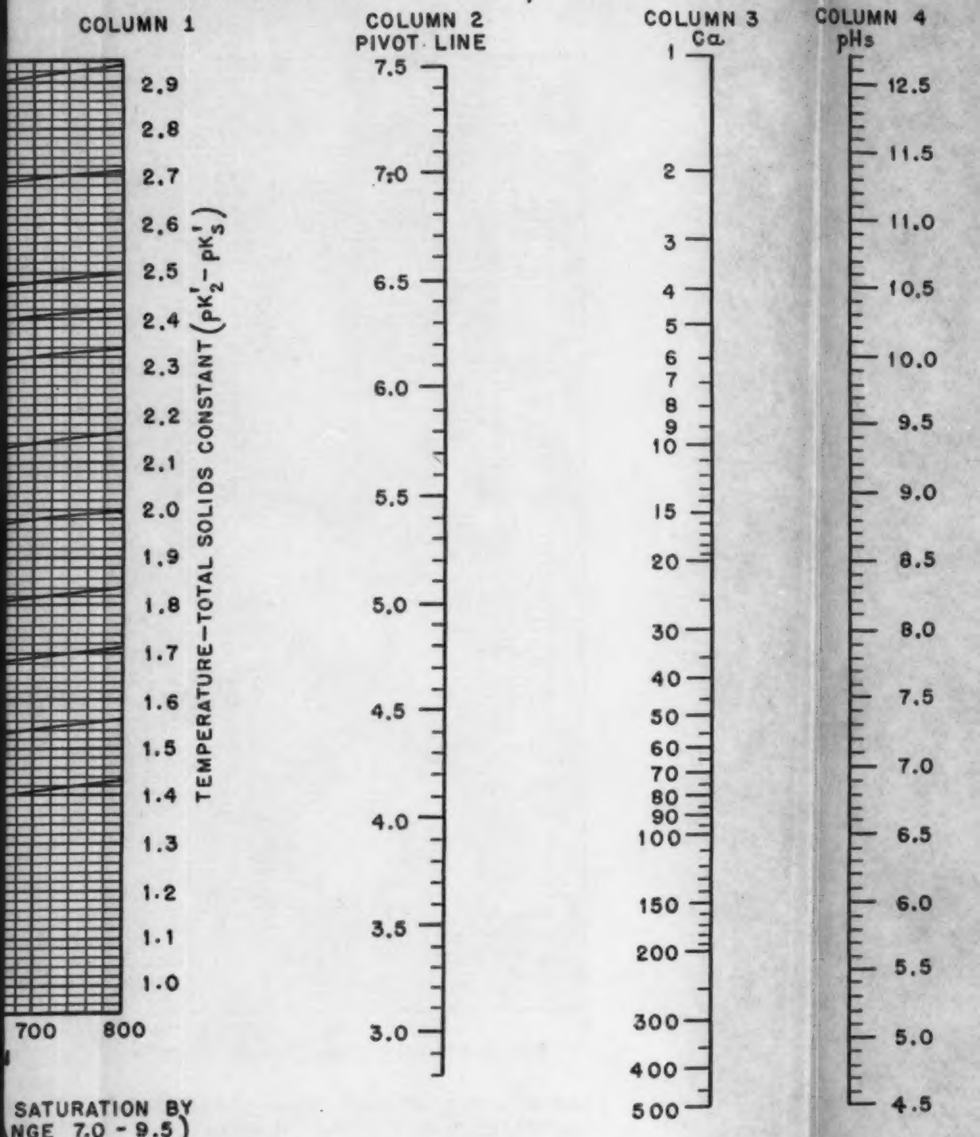
(a) KNOWING TEMPERATURE AND TOTAL DISSOLVED SOLIDS, FIND

(b) ALIGN THIS CONSTANT WITH GIVEN VALUE OF CALCIUM ON C

(c) ALIGN THIS POINT ON PIVOT LINE WITH GIVEN ALKALINITY

SATURATION INDEX IS pH ACTUAL MINUS pH SATURATION. E.G.

NOMOGRAM FOR DETERMINATION OF pH SATURATION BY LANGELIER'S



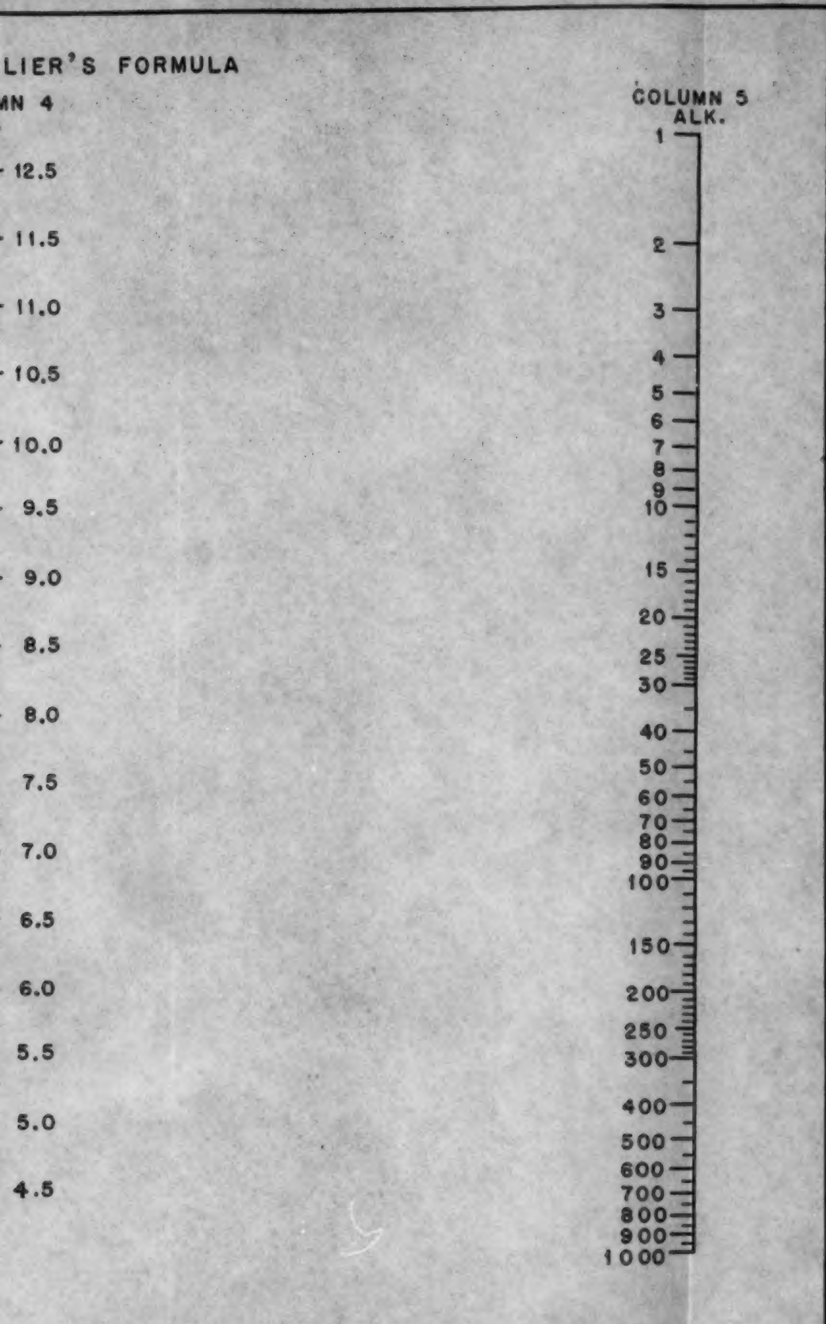
SATURATION BY
NGE 7.0 - 9.5)

F $CaCO_3$

LION.
WHICH pH SATURATION IS DESIRED.

D SOLIDS, FIND TEMPERATURE & TOTAL SOLIDS CONSTANT ON COL. 1
F CALCIUM ON COL. 3 OF CHART, THEN LOCATE POINT ON COL. 2 OF CHART (PIVOT
EN ALKALINITY ON COL. 5; READ pH SATURATION ON COLUMN 4.

URATION. E. G. --	pH ACTUAL.	pH SATURATION.	SATURATION INDEX.
	7.6	8.1	−0.5 (CORROSIVE)
	8.4	7.8	+ 0.6 (SCALE FORMING)



GRAPH AND NOMOGRAM FOR DETERMINATION OF pH SATURATION AND LANGELIER'S SATURATION INDEX BASED ON ARTICLE IN OCT. 1936, ISSUE OF AMERICAN WATER WORKS ASSOCIATION JOURNAL AND LATER CORRECTIONS FOR TABLES 2 AND 4. PREPARED FOR CHARLES P. HOOVER OF THE COLUMBUS, OHIO, WATER SOFTENING AND PURIFICATION PLANT BY MERRILL L. RIEHL SEPT. 17, 1938.

w

o
p
d
t

M
l
a
f
s

w
T
s
C
c
t
a
a
c
w
p
i

b
a
i
p
e
t
c
b
t

of corrosion. From now on a comparison of the index records on public supplies, together with a knowledge of their behavior in the distribution systems, should add materially to our knowledge as to what the index should be to produce acceptable results.

Mr. Langelier, in a private communication (commenting on Mr. Norcom's statement that, if anything is added to the water, such as lime or other alkali, a change in chemical constituents is brought about, and another complete analysis is required) suggested the following procedure for estimating the correct dosage of lime necessary for producing equilibrium:

"Divide a liter of the water into five equal parts. If it is a soft water, the equilibrium pH will be in the vicinity of say 8.5 to 9.5. Therefore select an indicator that falls within this range and add some to each 200 cc. portion of water. To the first jar add standard $0.02N$ $Ca(OH)_2$ from a burette until the alkaline color for that indicator is developed, noting also the minimum quantity of lime required to bring the sample into the range of the indicator. This will give a fair estimate of the approximate quantity of lime required. Now add varying amounts of lime (between the limits indicated) to the other four test portions. If the sample has been run for Ca^{++}, it will be possible to compute pCa for each test portion. Now run pAlk and pH on each. The pH saturation is quickly and accurately indicated by interpolation."

Thomas H. Wiggin (August, 1938 Journal of the A. W. W. A., p. 1351) reports that internal corrosion of pipes is very prevalent—almost universal. He further says that "Chemical treatment according to current theories doubtlessly prevents red water troubles, but lime treatment has proved of only limited value in the writer's experience in preserving flow coefficients." He expresses the hope that those who have measured coefficients in connection with chemical treatment will come forward with information and that those who have rested with confidence in chemical theory will make flow tests to see if their confidence is justified.

We have now, it seems to me, developed all the methods and techniques necessary to make an intelligent study of the relation of chemical balance to internal corrosion of pipes. The relationship of pH saturation indices to pipe flow coefficients, as shown by a large number of observations of actual installations, would undoubtedly contribute materially to our knowledge of anti-corrosion water treatment.

Discussion by W. F. LANGELIER.* I feel that the nomogram for computing the saturation index is very ingenious and should be useful to chemists who have much of this work to do. It is superior to the combination of my figure 1 and table 2 in the original paper, and I am pleased to see it published.

Shortly after publication of my paper, Mr. E. Gelbaum of the technical department of the Permutit Company discovered that I had made a slight error in my computations of tables 2 and 4 in so far as they relate to values of pK'_s at temperatures other than 25°C. In these computations I used the equation, $pK'_{st} = pK'_{s(25°C)} - \log r$, whereas I should have used $pK'_{st} = pK'_{s(25°C.)} - 2 \log r$. Mr. Hoover and Mr. Riehl have used corrected data in preparing the above paper and nomogram. It will be noted that the corrected values lead to a somewhat more pronounced temperature effect, but it may be open to question whether the corrections are significant in view of the other assumptions that I was forced to make and which are discussed on page 1512 of the original paper (Jour. A.W.W.A., **28**: 1500 (Oct. 1936)).

In regard to the matter of the relative practicability of the simple marble test as described by Mr. Hoover and the Saturation-Index test, I should like to point out that the two tests differ in much the same way as the residual-chlorine test differs from the chlorine-demand test. The marble test is intended to determine qualitatively whether a given water is in balance, whereas the Index test should accomplish this and in addition should indicate quantitatively the dosage necessary to attain balance. I should say that the marble test should supplement the Index test rather than displace it. It should be noted, however, that the principle of the marble test can also be used to estimate dosage. P. L. McLaughlin has developed such a procedure.

The method for carrying out this Index test which I have communicated to Mr. Hoover can be completed easily within an hour, provided the initial calcium content of the water is known.

In reading several discussions of the Index method of control, which have appeared in various publications, I have been impressed by an apparent demand for simplicity even at the expense of accuracy. Doubtless there are situations where a dependable semi-quantitative test would be adequate. To meet this demand, I should recommend

*Assoc. Prof. San. Eng., University of California, Berkeley, Calif.

a test which is even simpler and more positive than the marble test. Briefly, this consists of adding gradually increasing amounts of the diluted dosing chemical to several 100 cc. portions of water in Nessler tubes. After thorough mixing and after standing for one hour, the well illuminated tubes are examined against a dark background for visual evidences of opalescence or turbidity caused by the precipitation of calcium carbonate. From these observations the minimum or correct dosage for balance is readily computed. This is probably the simplest of all procedures for estimating correct dosage, and in our laboratory we have found that this method checks the results obtained by other methods.

Finally, I should like also to point out that the term "pH saturation of water," as it has been used in discussion, is confusing, since it implies that an unsaturated water may have only one "saturation pH." Actually, the "saturation pH" will be different depending upon what chemical is used to bring the water into balance. This is because, for any given water being brought into balance, there are three variables—calcium, alkalinity, and pH, each of which may assume a different value depending upon the chemical used. Thus for a given water the "saturation pH" obtained in the marble test will differ from the "saturation pH" which results from the addition of lime. In my original paper, I used the term "saturation pH" as a hypothetical quantity, i.e., the pH at equilibrium, *assuming no change in composition*. This hypothetical pH_s approaches the true pH_s as the Saturation Index approaches zero.

NEW PLATING MEDIUM FOR COLIFORM ANALYSIS CITRATE RICINOLEATE AGAR

I. DEVELOPMENT AND THEORY

By M. L. Littman and C. N. Stark

Following the report by Stark and England (1) on the use of sodium ricinoleate as a specific inhibitor for non-coliform organisms in formate-ricinoleate broth, studies were begun on the development of a plating medium incorporating this compound.

Larson (2) observed that intestinal bacteria grew well in a medium having a low surface tension, that pneumococci and streptococci were suppressed by a surface tension of 50 dynes and that the surface tension limit for the growth of *Bacillus anthracis* was sharp at 46 dynes. Ayers, Rupp and Johnson (3) found that *Escherichia coli* and *Aerobacter aerogenes* grew well when the surface tension was depressed to 35 dynes with sodium ricinoleate and concluded that generally the growth of streptococci was prevented by reducing the surface tension to 40–41 dynes. Wolf (4) observed *Escherichia coli* to be relatively indifferent to a surface tension sufficiently low so that it definitely inhibited anaerobes. Walker (5) found that meningococci, gonococci, streptococci and diphtheria organisms were readily killed by soaps like sodium laurate, potassium myristate and coconut oil soaps, but that dysentery, paratyphoid and typhoid organisms were resistant to the soaps of the unsaturated acids (as is ricinoleic) but were killed by the soaps of the saturated acids. Frobisher (6) noted that low surface tensions inhibited the streptococci, staphylococci and pneumococci; but not *Escherichia coli, Aerobacter aerogenes, Eberthella typhi* or *Bacterium morgani.* Albus (7) observed that the depressant action of sodium ricinoleate on the growth of *Escherichia coli* was not evident after the logarithmic growth phase had passed. Koslowski (8) showed that streptococci isolated from cases of erysipelas, measles and scarlet fever were easily killed in 7

A record of research done by M. L. Littman and Dr. C. N. Stark, Laboratory of Bacteriology, Cornell University, Ithaca, N. Y.

hours by 0.02 per cent sodium ricinoleate in 0.02 per cent glucose meat infusion broth. He reported that *S. lactis*, *S. pyogenes*, *S. viridans*, *Corynebacterium diphtheriae* and *Mycobacterium tuberculosis* were inhibited by 0.02 per cent sodium ricinoleate; and that *Escherichia coli*, *Salmonella paratyphi*, *Shigella dysenteriae* and *Streptococcus fecalis* were resistant to the bactericidal and inhibitory action of the soap.

Stark and England (1) were the first to apply sodium ricinoleate to coliform analysis of water and milk. These investigators used sodium formate, lactose and peptone in their broth for growth stimulation of coliform organisms and to cause the production of large amounts of gas. Sodium ricinoleate was used to inhibit the gas production of the false test organisms studied. It was noted that 0.1 per cent sodium ricinoleate when combined with 0.5 per cent sodium formate, 0.5 per cent lactose and 0.5 per cent peptone was the maximum amount of soap it was possible to add without affecting gas production of the coliform group. The recommended composition of the broth was as above.

DIFFICULTIES MET IN EXPERIMENTAL WORK

At the beginning of the research to produce a plating medium containing sodium ricinoleate, several difficulties were encountered and overcome as follows:

1. Divalent ions normally present in the agar precipitated a small part of the 0.1 per cent sodium ricinoleate present, as insoluble ricinoleates. The resultant precipitation produced a sprinkling of tiny white specks resembling pin point colonies throughout the agar. Dialysis of the agar to remove these interfering ions was considered but not resorted to because of the costliness of the process.

The introduction of neutral red in concentration of 1/20,000 eliminated the difficulty by staining the agar containing the soap a uniform, clear, blood red at pH 7.0. Neutral red was favored because of its value also as a pH indicator, since under aerobic conditions, it is red at pH 6.8 and lower and yellow at pH 8.0.

2. Another difficulty was the production of gas from lactose by the coliform organisms with the subsequent appearance of innumerable gas bubbles in the agar. This condition was considered extremely undesirable as it rendered counting of colonies difficult.

Since Pakes and Jollyman (9) reported that the presence of sodium

TABLE 1

Relative non-toxicity of citrate ricinoleate agar[1] containing 10 per cent skim milk to Escherichia organisms

ORGANISMS	NUMBER OF COLONIES IN: Nutrient agar			Citrate ricinoleate agar		
	Deep	Surface	Total	Deep	Surface	Total
Escherichia coli #71	112	11	123	112	2	114
Escherichia coli #52	149	10	159	128	5	133
Escherichia communior #69	220	17	237	218	6	224
Totals	481	38	519	458	13	471

[1] Colony counts on nutrient and citrate ricinoleate agars represent an average of triplicate plates. The ten millionth dilution plates were used in the counting. Composition of the citrate ricinoleate agar was: 0.5 per cent peptone, 0.3 per cent sodium citrate, 0.2 per cent sodium nitrate, 0.1 per cent sodium ricinoleate, 1.5 per cent agar, 1/20,000 neutral red, pH 7.0. Addition of 10 per cent skim milk provided 0.5 per cent lactose.

TABLE 2

Toxicity of citrate ricinoleate agar[1] containing 10 per cent skim milk to Aerobacter organisms

Organisms	NUMBER OF COLONIES IN: Nutrient agar			Citrate ricinoleate agar		
	Deep	Surface	Total	Deep	Surface	Total
Aer. aerogenes #67	150	48	198	72	23	95
Aer. aerogenes #53	112	21	123	86	11	97
Aer. cloacae #68	128	110	238	130	27	157
Aer. oxytocum #32	80	22	102	89	19	108
Aer. viscosum capsulatum #57	21	19	40	27	5	32
Totals	491	220	701	404	85	489

[1] Colony counts on nutrient and citrate ricinoleate agars represent an average of triplicate plates. The ten millionth dilution plates were used in the counting. Composition of the citrate ricinoleate agar was: 0.5 per cent peptone, 0.3 per cent sodium citrate, 0.2 per cent sodium nitrate, 0.1 per cent sodium ricinoleate, 1.5 per cent agar, 1/20,000 neutral red, pH 7.0. Addition of 10 per cent skim milk provided 0.5 per cent lactose.

nitrate prevented the formation of gas from sodium formate by the coliform organisms, experiments were conducted both to confirm and apply these findings. The inhibition of gas formation with so-

dium nitrate was confirmed by us. The biochemical phenomenon was represented by Pakes and Jollyman as:

$$HCOONa + NaNO_3 = NaHCO_3 + NaNO_2.$$

The next step followed was the addition of sodium nitrate to lactose agar to test for inhibition of gas formation. The composition of the agar used was: 0.5 per cent sodium formate, 0.5 per cent lactose, 0.5 per cent peptone, 0.1 per cent sodium ricinoleate, 1.5 per cent agar, 1/20,000 neutral red, pH 7.0. Three different rapidly growing, 24-hour-old cultures of *Escherichia coli* and seven different strains of *Aerobacter aerogenes* were plated on this agar which contained in addition from 0.65 per cent to 0 per cent sodium nitrate.

It was observed that the addition of as little as 0.1 per cent sodium nitrate prevented gas production in the agar with no deleterious effect on colony size or growth. The same phenomenon occurred in agar identical with the above except that sodium formate was eliminated. The presence of sodium nitrate also insured aerobiosis to enable neutral red to function properly as a pH indicator.

3. A third difficulty encountered was the toxicity of sodium ricinoleate to the coliform organisms. The addition of 0.1 per cent sodium ricinoleate to peptone-lactose-sodium citrate-sodium nitrate-neutral red agar reduced the numbers of *Esch. coli* 50 per cent and those of *Aer. aerogenes* 43 per cent. (Sodium citrate was substituted for sodium formate to make available a means of differentiating Escherichia colonies from those of Aerobacter and will be discussed more fully later.)

Several attempts to reduce the toxicity of sodium ricinoleate to coliform organisms met with failure (Littman (10)). It was noted finally that the introduction of approximately 10 per cent skim milk to the agar decreased the toxicity of the medium for *Esch. coli* from 50 per cent to 9.2 per cent reduction of numbers (table 1) and for *Aer. aerogenes* from 43 per cent to 30 per cent reduction (table 2).

The cause of the detoxifying action of skim milk on sodium ricinoleate is problematical. Although divalent salts introduced with the milk will cause insoluble ricinoleates to form, nevertheless the amount of salts thus added is small and can produce only a very small reduction in the actual concentration of the soluble soap. When the concentration of sodium ricinoleate in the citrate ricinoleate agar, not containing skim milk, was reduced from 0.1 per cent to 0.05 per cent, the reduction in numbers of *Esch. coli* fell from 50 per cent to only 33

per cent reduction. This indicates that the detoxifying action of skim milk on sodium ricinoleate is not a question of reduction in concentration of the soap.

With Escherichia as the test organism (table 1) in citrate ricinoleate agar containing skim milk, the decrease in numbers caused by the soap at the surface of the agar accounted for one-half of the total reduction of 9.2 per cent, while the soap in the depths of the agar accounted for the other half.

With Aerobacter as the test organism, the decrease in numbers caused by the sodium ricinoleate at the surface of the agar accounted for two-thirds of the total reduction of 30 per cent, while the soap in the depths accounted for one-third.

TABLE 3

Inhibition of colony formation by citrate ricinoleate agar

Clostridium sporogenes	*Sarcina lutea*
Clostridium welchii	*Staphylococcus aureus*
Bacillus cereus	*Staphylococcus albus*
Bacillus megatherium	*Rhodococcus Sp.*
Bacillus mesentericus	*Lactobacillus acidophilus*
Bacillus mycoides	*Lactobacillus bulgaricus*
Bacillus subtilis	*Lactobacillus casei*
Bacillus peptogenes	*Achromobacter viscosum*
Streptococcus lactis	*Erwinia caratovora*
Streptococcus fecalis	*Vibrio comma*

The high "unit area" concentration of the sodium ricinoleate at the agar surface was responsible for a considerable part of the total reduction in numbers of coliform organisms. Attempts to reduce further this surface toxicity of the soap by layering agar solutions over the surface of the medium met with failure.

Sodium ricinoleate in concentration of 0.1 per cent was used to inhibit the colony formation of false test organisms. To observe this property, rapidly growing, 24-hour-old cultures of the organisms listed in table 3 were plated out in different dilutions on two citrate ricinoleate agars* and on a standard agar medium and environment

* Agar 1: 0.5 per cent peptone, 0.3 per cent sodium citrate, 0.2 per cent sodium nitrate, 0.1 per cent sodium ricinoleate, 1.5 per cent agar, 1/20,000 neutral red, pH 7.0, 10 per cent skim milk providing 0.5 per cent lactose.

Agar 2: Same as Agar 1 except that 0.5 per cent lactose was substituted for skim milk.

suitable for the growth of each of the organisms tested. All of the organisms listed below produced colonies on the standard media but failed to produce visible colonies at any dilutions in the citrate ricinoleate agars in 72 hours at 37°C.

USE OF SODIUM CITRATE

Koser (11) in an extensive study of the utilization of organic acids by coliform organisms, obtained striking results with citric acid. With this acid as the sole source of carbon and ammonium phosphate as the source of nitrogen, he observed that "fecal *Esch. coli* cultures consistently failed to develop while *Aer. aerogenes* cultures multiplied readily, produced luxuriant growth, and changed the pH of the medium from 6.7 to pH 8.4–9.0. The *Esch. coli* cultures produced no change in pH."

The ability of *Aer. aerogenes* to utilize sodium citrate as the source of carbon with resulting alkaline change, and failure of *Esch. coli* to do so, was made use of in the citrate ricinoleate medium. Sodium citrate is not used as the sole source of carbon in the citrate ricinoleate agar since peptone, lactose and milk proteins are also present. It is used instead as a means of producing colonies of *Aer. aerogenes* different in appearance from those of *Esch. coli*.

Optimum concentration of sodium citrate was observed to be 0.3 per cent which was the least amount needed to provide maximum alkaline change by Aerobacter in citrate ricinoleate agar.

USE OF INDICATORS

In addition to its use in masking the insoluble ricinoleate particles in the agar, neutral red was also used with brom thymol blue to detect acid or alkaline changes in the medium. This combination of indicators was found to give excellent results. A mixture of neutral red and brom thymol blue in equal concentrations of 1/20,000 is red in the acid range and blue in the alkaline range of pH values. Neutral red changes from red to yellow from pH 6.8–8.0, while brom thymol blue changes from yellow to blue from pH 6.0–7.6. On the acid side, below pH 6.8 the predominating color of the mixture of indicators is red since the red color of the neutral red masks the yellow color of the brom thymol blue. Conversely, on the alkaline side above pH 7.6, the predominating color is blue since the blue color of the brom thymol blue masks the yellow color of the neutral red.

The optimum concentration of both indicators was found to be the

same, at 1/20,000. This was the lowest concentration of indicator producing a satisfactory color change. The neutral red optimum was judged from the red color change which acid forming colonies of Escherichia produced on the agar. The brom thymol blue optimum was selected from the blue color change produced by sodium citrate utilizing Aerobacter colonies.

The non-toxicity of the double indicators (each in concentration of 1/20,000) was measured in nutrient agars with and without the indicators. The test organisms were 24-hour-old cultures of *Esch. coli, Aer. aerogenes, Eberthella typhi, Proteus vulgaris* and *Salmonella paratyphi*. It was noted that the presence of the indicators in the afore-mentioned concentration exhibited no noticeable effect on the total count and colony size of the organisms studied. Citrate ricinoleate agar was not used as the testing medium because of the difficulty of accurately counting colonies on this medium, without indicators, particularly in the absence of neutral red.

The non-toxicity of neutral red in a concentration of 0.005 per cent or 1/20,000 on the growth of *Esch. coli* was reported by Knaysi (12). An agglomeration of neutral red occurred around acid forming colonies to produce red colonies much larger in apparent size than actual. This increase was attributed to both acid production by the coliform colony and physical adsorption of the indicator. Experiments were conducted which demonstrated that the presence of sodium ricinoleate in citrate ricinoleate agar was essential for extensive adsorption to occur. The absence of sodium ricinoleate in the agar caused a very marked decrease in the apparent size of lactose fermenting coliform colonies.

COMPOSITION OF MEDIUM

In spite of the presence of a considerable amount of milk protein in citrate ricinoleate agar, the addition of 0.5 per cent peptone was, nevertheless, found to be necessary for the production of larger coliform colonies and for better utilization of sodium citrate by Aerobacter organisms.

Tryptone was substituted for peptone in the citrate ricinoleate agar and found to be unnecessary as it neither increased the size of coliform colonies nor intensified the alkaline change produced by Aerobacter.

For coliform analysis of water, the medium is composed as shown in table 4.

For coliform analysis of milk, the composition is different. Skim milk powder and lactose are omitted from the medium since the introduction of 1 cc. of milk sample to each petri dish before the agar is poured compensates for these materials as it accomplishes the following: (a) introduces bacterial flora, (b) introduces 0.45 per cent lactose, and (c) introduces milk which reduces the toxicity of sodium ricinoleate to coliform organisms.

The intestinal organisms capable of producing colonies on citrate ricinoleate agar fall into three distinct groups (table 5). Group 1 is composed of Escherichia and Eberthella organisms and produce red colonies in 24 hours which remain unchanged with prolonged incubation. This group is non-proteolytic and produces no alkaline change from sodium citrate in the medium.

TABLE 4

Composition of medium

0.5 per cent peptone
0.3 per cent sodium citrate
0.2 per cent sodium nitrate
0.1 per cent sodium ricinoleate
0.2 per cent lactose
1.5 per cent shredded agar
0.6 per cent skim milk powder
0.005 per cent neutral red
0.005 per cent brom thymol blue

Group 2 includes Aerobacter, Salmonella and Shigella organisms and produce red colonies in 24 hours which may either possess blue rings or be entirely blue depending upon the activity of the colony. This group is non-proteolytic and does produce an alkaline change from sodium citrate in the medium. The fermentative ability of Citrobacter organisms on sodium citrate classes these bacteria with Aerobacter.

Group 3 consists of Proteus, Serratia, and Pseudomonas organisms. Proteus and Serratia produce red colonies in 24 hours which may also either possess blue rings or be entirely blue. Pseudomonas is usually blue green in 24 hours. This group is markedly proteolytic and produces an alkaline change in the medium.

Since citrate ricinoleate agar for both water and milk analysis contains milk proteins, detection of proteolysis is possible by flooding the plates with a protein precipitant like acidic 15 per cent mercuric

chloride (Frazier (13)), and allowing to react for 15 to 30 minutes. Proteolysis is indicated by the presence of a clear zone around the colony. Visible observation of proteolysis without use of protein precipitants is unreliable since vigorous acid producing coliform organisms which are non-proteolytic to milk casein may show cleared

TABLE 5

Classification of intestinal organisms on citrate ricinoleate agar after 24 hours incubation at 37°C.

ORGANISMS	COLOR OF COLONY	ALKALINE CHANGE[4]	CASEIN PROTEOLYSIS[5]	APPARENT AVERAGE COLONY SIZE[6] MM.
Group 1				
Escherichia coli[1]	Red[3]	−	−	3.0
Escherichia communior	Red[3]	−	−	3.0
Eberthella typhi	Red[3]	−	−	0.4
Group 2				
Aerobacter aerogenes[2]	Red, blue green ring	+	−	3.0
Salmonella paratyphi	Red, blue green ring	+	−	2.0
Salmonella schottmulleri	Red, blue green ring	+	−	2.0
Salmonella enteritidis	Red, blue green ring	+	−	2.0
Shigella paradysenteriae	Red, blue green ring	+	−	2.0
Group 3				
Proteus vulgaris	Red, blue green ring	+	+	2.5
Serratia marcescens	Red, blue green ring	+	+	2.5
Pseudomonas pyocyaneus	Blue green	+	+	2.0
Pseudomonas fluorescens	Blue green	+	+	2.0

[1] Two strains.

[2] Seven strains including *Aer. levans, Aer. cloacae, Aer. oxytocum, Aer. capsulatum.*

[3] Remains unchanged with prolonged incubation.

[4] Signified in the presence of brom thymol blue either by production of blue or blue green ring about colony or by blue or blue green colony.

[5] Detected by flooding plate with acid solution of 15 per cent mercuric chloride. Clear zone around colony indicates casein proteolysis.

[6] On uncrowded plates.

zones due to a dissolving of suspended ricinoleates in the immediate vicinity of the colony. Precipitation of the protein in the agar by afore-mentioned materials obscures these zones entirely and allows only proteolytic colonies to exhibit cleared areas.

The "Escherichia" count of a sample of water or milk is that number of red colonies which exhibit neither alkaline nor proteolytic

changes on the medium. The "Aerobacter" count is that number of colonies which are red with blue green rings or are entirely blue green, but which exhibit no proteolytic action in the medium. The "Non-coliform" count is that number of colonies exhibiting simultaneously both alkaline and proteolytic changes in the medium.

A recommended procedure for counting plates is as follows: Count the number of red colonies not exhibiting alkaline changes. This is the "Escherichia" count. Count the remainder of the colonies, then flood plate with a protein precipitant to count the number of proteolytic colonies. The second count minus the number of proteolytic colonies is the "Aerobacter" count.

TABLE 6

Cultural characteristics of "Escherichia" colonies

NUMBER OF CULTURES	LACTOSE BROTH	METHYL RED	VOGES-PROSKAUER	CITRATE BROTH
108	AG[1]	+	−	−
7	AG	+	+	−
2	AG	−	+	+
1	AG	−	+	−
2	A	+	−	−
1	A	+	−	+
1	A	+	+	−
Totals 122	118 AG	119 +	111 −	119 −
	4 A	3 −	11 +	3 +

[1] Acid and gas.

In the counting of citrate ricinoleate agar plates, the substitution of a white cardboard for the black background of the plate counting device is recommended.

APPLICATION OF MEDIUM

Several hundred samples of pasteurized milk were examined with citrate ricinoleate agar with the above technique and the utilization of the medium for the counting of coliform organisms established (Littman and Stark (14)). From 30 agar plates representing an equal number of milk samples, 122 pure cultures were picked from typical, red, well isolated "Escherichia" colonies. The cultures were then studied in lactose broth, methyl red media, citrate broth and for the Voges-Proskauer test. The results are presented in table 6. It

will be observed that 119 out of 122 colonies or 97.5 per cent confirmed as Escherichia according to the methyl red and citrate tests. The three citrate positive cultures were plated out on citrate ricinoleate agar and proved to be slow citrate fermenting Aerobacters.

The use of citrate ricinoleate agar in the coliform analysis of water and sewage was under study during the last year by the senior author at the New Jersey Agricultural Experiment Station, New Brunswick, New Jersey. These results are presented as the second part of this paper.

The ability of citrate ricinoleate agar to make a direct "Escherichia" count entirely separate from the "Aerobacter" count provides a much needed, simple and rapid means for the sanitary examination of shellfish.

PREPARATION OF MEDIUM FOR WATER ANALYSIS

To prepare 1 liter of single strength citrate ricinoleate agar dissolve the following in 900 cc. distilled water: 5 grams peptone (S_1), 3 grams anhydrous sodium citrate (S_2), 2 grams sodium nitrate (S_3), 1 gram sodium ricinoleate (S_4), 2 grams lactose (S_5), 15 grams shredded agar.

The mixture is heated in the Arnold to dissolve the agar completely, then filtered through "agar filter" paper or a pad of cheese cloth. Six grams skim milk powder (S_6) dissolved first in 60 cc. hot distilled water is next added and shaken to distribute. Fifty milligrams or 0.05 grams of brom thymol blue (S_7) is first dissolved in 1.0 cc. N/10 NaOH to form the mono-sodium salt and then made up to 20 cc. with distilled water. This indicator solution is then added to the agar mixture and distributed. The color of the mixture at this point should be deep green. No pH adjustment of the medium is necessary because of the buffering action of the peptone. Fifty milligrams of neutral red (S_8) first dissolved carefully in 20 cc. distilled water are next added and distributed throughout the medium. The agar medium is then distributed in small flasks, sterilized at 15 pounds pressure for 15 minutes, cooled, stored in the refrigerator and remelted when needed. Several months storage of the medium at refrigerator temperatures is possible without deterioration or excessive drying.

To prepare double strength agar for the plating of 10 cc. samples of water, all ingredients of the medium, except water are doubled. Method of preparation is identical with that of single strength agar.

Approximately 10 cc. of double strength agar is poured at ± 50°C. into petri dishes containing 10 cc. water sample and mixed. An alternate method is the preliminary warming of the petri dish and its contents over an open flame and pouring the agar at ± 46°C This method must be performed cautiously.

Single strength agar is used for samples 1 cc. and smaller. The hardened medium in the petri dish appears clear, homogenous, blood red. The plates are incubated at 37°C. for 24 hours and the complete count made as recommended.

Longer incubation periods and different temperatures may be used, if desired, in making a differentiation of Aerobacter from Escherichia organisms. The separation of these two groups on citrate ricinoleate agar plates at the end of 24 hours incubation at 37°C. is not 100 per cent efficient, since any "slower than average" citrate fermenting Aerobacters may require more than this time of incubation to produce discernible blue zones about their colonies. Over-incubation is to be avoided, since the whole plate may turn blue if there is a large number of active citrate fermenting Aerobacters present. Twenty-four hours of incubation at 37°C. was selected as the best time of observation for best differentiation of Aerobacter. Evidence of slow citrate fermenting Aerobacters was found in the study of pure cultures picked as Escherichia from citrate ricinoleate agar plates (table 5). Three of the 122 "Escherichia" cultures proved to be slow citrate fermenting Aerobacters. Since biochemical characteristics of groups of organisms are known to vary in degree among several strains of a single species, the differentiation of Aerobacter from Escherichia on this medium cannot be an absolute test, but may approach a very high efficiency under ideal conditions.

The method of preparation of the medium for milk analysis is identical with that for water analysis but the composition is slightly different. Skim milk powder and lactose are omitted from the medium and the starting volume of distilled water is 960 cc. The medium thus prepared is planned to receive a 1 cc. sample of milk to be examined.

LIST OF SPECIFICATIONS

(S_1) Bacto

(S_2) C.P. use 0.342% $Na_3C_6H_5O_7 \cdot 2H_2O$ or 0.416% of $Na_3C_6H_5O_7 \cdot 5\frac{1}{2}H_2O$

(S_3) C. P. Baker's

(S_4) Eastman Kodak's Powdered Practical, Lot No. P1779

(S_5) C. P. Baker's Powdered

(S_6) Good grade, fresh

(S_7) LaMotte's Powdered

(S_8) C.I. No. 825 (Ehrlich) Schultz No. 670

There are some precautions which should be taken. Both indicators should be dissolved well; no solid particles of undissolved dye should be left.

REFERENCES

1. Stark, C. N. and England, C. W. Jour. Bact. **29**: 26, (1935).
2. Larson, W. P. Abstracts Bact. **5**: 2, (1921).
3. Ayers, S. H., Rupp, P. and Johnson, W. T., Jr. Jour. Inf. Dis. **33**: 202 (1923).
4. Wolf, C. G. L. Jour. Biochem. **17**: 813 (1923).
5. Walker, J. E. Jour. Inf. Dis. **7**: 73 (1910).
6. Frobisher, M., Jr. Jour. Inf. Dis. **38**: 66 (1926).
7. Albus, W. R. Jour. Bact. **13**: 7 (1927).
8. Koslowski, A. Jour. Bact. **16**: 203 (1928).
9. Pakes, W. C. C. and Jollyman, W. H. Proc. Chem. Soc. London, **17**: 29 (1901).
10. Littman, M. L. Thesis, Cornell University.
11. Koser, S. A. Jour. Bact. **8**: 493 (1923).
12. Knaysi, G. Jour. Bact. **30**: 193 (1935).
13. Frazier, W. C. Jour. Inf. Dis. **39**: 302 (1926).
14. Littman, M. L. and Stark, C. N. Jour. Bact. **34**: 348 (1937).

II. APPLICATION TO WATER AND SEWAGE

By M. L. Littman

With the development of citrate ricinoleate agar as described in part I (also reference 1, this paper), this medium was studied for its possible use as a rapid routine test for coliform analysis of potable and polluted waters.

Samples of water received at the laboratory for coliform analysis were examined in parallel with the standard lactose test and with citrate ricinoleate agar. Potable waters were inoculated in five 10, 1 and 0.1 cc. amounts into lactose broth and in five 10 cc. amounts in double strength citrate ricinoleate agar. Positive presumptive tests in lactose broth were confirmed in brilliant green bile broth and the coliform count secured with "most probable number" method. All observations with lactose broth conformed with Standard Methods (2).

Only five 10 cc. inoculations of potable water into citrate ricinoleate agar were needed to count the same number of organisms detected by 15 lactose broth tubes receiving five 10, 1 and 0.1 cc. amounts of water since the range in numbers of organisms covered by the agar was equivalent to that covered by lactose broth. Ten cc. of double strength citrate ricinoleate agar was used with 10 cc. of potable water sample, while single strength agar was used for appropriate dilutions of river water and sewage. This medium was prepared according to the procedure set forth in the preceding part of this paper. Observations of the citrate ricinoleate agar plates were completed in 24 hours. The Escherichia and Aerobacter groups were counted together by subtracting the number of "non-coliform" proteolytic colonies from the total number according to the classification outlined in the previous part.

Seventy-five samples of potable water were examined. Results are presented in table 1. Pure cultures of coliform colonies isolated

A Journal Series paper, New Jersey Agricultural Experiment Station, New Brunswick, N. J., by M. L. Littman, graduate student, Department Water and Sewage Research.

from citrate ricinoleate plates of samples 11, 12, 13, 16 and 19 were found to be predominantly slow gas forming organisms. The presence of these organisms in the samples under question indicates the probable cause of their very low count in lactose broth as compared to citrate ricinoleate agar. Omission of samples 11 and 18 from the count produced an average coliform content of 37/100 cc. water in lactose broth against 48/100 cc. water in citrate ricinoleate agar. Discounting samples 11 and 18 whose effect on average count would be minimized if a much greater number of water samples had been analyzed, there is a 30 per cent greater productivity of citrate ricinoleate agar over lactose broth in the actual numbers of coliform organisms.

The productivity correlation of the two media is presented in table 2, where is shown a 10 per cent greater productivity of citrate ricinoleate agar over lactose broth in the number of potable water samples found to contain coliform organisms. Of the 32 positive samples, lactose broth failed to detect coliforms in 3 of the positive samples or 9 per cent, while citrate ricinoleate agar failed to detect coliforms in 1 of the positive samples or 3 per cent. Of the 28 samples positive in both media, 72 per cent showed higher counts in citrate ricinoleate agar and 28 per cent showed lower counts in citrate ricinoleate agar than in lactose broth.

Forty-seven samples of river water obtained from various points on the Raritan River, N. J., were examined with lactose broth and with citrate ricinoleate agar. Table 3 presents these results. Average coliform count in citrate ricinoleate agar was 96.1/cc. compared with 88/cc. in lactose broth. Table 4 compares further the productivity of the two media. In the case of river water which contains many more coliform organisms than the potable waters there occurs a 9 per cent greater productivity of citrate ricinoleate agar over lactose broth in the actual number of coliform organisms, but a 4 per cent lower productivity of the agar medium to lactose broth in the number of river water samples detected to contain coliform organisms. Of the 45 positive samples, lactose broth failed to detect coliforms in 1 of the samples or 2 per cent, while citrate ricinoleate agar failed to detect in 3 of the samples or 7 per cent. Of the 41 samples positive in both media however, 63 per cent of these showed higher counts and 36 per cent showed lower counts in citrate ricinoleate agar than in lactose broth.

Results from coliform analysis of six samples of raw sewage are

TABLE 1

Examination of potable waters

SAMPLE NUMBER	SOURCE	COLIFORM COUNT PER 100 CC. WATER: Lactose broth	COLIFORM COUNT PER 100 CC. WATER: Citrate ricinoleate agar
1	Driven well	8	6
2	Driven well	2	2
3	Artesian well	4	0
4	Driven well	6	4
5	Driven well[1]	0	4
6	Driven well	2.6	9.5
7	Artesian well	11.0	30.
8	Pool	50	20
9	Shallow well	60	60
10	Driven well	420	440
11	Driven well[2]	35	630
12	Driven well[1]	0	30
13	Driven well[1]	0	27
14	Well	350	470
15	Well	25	27
16	Well	8	42
17	Well	50	82
18	Well[2]	12	354
19	Well[2]	4	70
20	Ocean	8	20
21	Dug well	80	60
22	Artesian well	2	3
23	Well	250	210
24	Well	130	430
25	Well[2]	13	83
26	Well	250	300
27	Well	250	300
28	Well	250	340
29	Well	130	100
30	Spring	350	410
31	Lake	50	46
32	Well	5	3
33	Ocean	0	0
34-75	Wells[3]	0	0
Average count......................................		37.6	61.5
Average count[4]...................................		37.0	48.3

[1] Coliform isolated from CR plates produced AG in lactose broth in 4–6 days at 37°C.

[2] Further incubation of negative lactose tubes beyond 48 hours produced additional positive tubes to raise the count in lactose broth.

[3] Forty-two samples.

[4] Samples 11 and 18 omitted from average.

listed in table 5. Average coliform count in lactose broth was 34,600,000/cc. against 31,700,000/cc. in citrate ricinoleate agar. These higher counts obtained with lactose broth in sewages very high in coliform organisms are likely due to the empirical inaccuracies of the "most probable number" method. In the examination of a potable water with five tubes of 10, 1 and 0.1 cc. amounts, a change in the significant number from 542 to 543 will change the most probable number from 25/100 cc. to 30/100 cc. This change is small. In the examination of sewage, however, a similar change of the significant number will change the most probable number from 25

TABLE 2

Productivity correlation of lactose broth with citrate ricinoleate agar

DESCRIPTION OF SAMPLES	NUMBER OF SAMPLES
Total examined	75
Coliform negative	43
Coliform positive in either media	32
Positive in lactose broth	29
Positive in CR[1] agar	32
Positive in CR agar	100 per cent
Positive in lactose broth	90 per cent
Higher productivity of CR agar to lactose broth	10 per cent
Positive in both media	28
Positive in lactose broth, negative in CR agar	1
Negative in lactose broth, positive in CR agar	3
Higher count in lactose broth than in CR agar	8
Higher count in CR agar than in lactose broth	20
Total positive showing higher count in lactose broth than in CR agar	28 per cent
Total positive showing higher count in CR agar than in lactose broth	72 per cent

[1] Citrate ricinoleate.

million/cc. to 30 million/cc. with dilutions of 1, 10, and 100 million. The validity of the last figure in the significant number thus may be questioned because of its dependence upon the implantation of one or more coliform organisms in the last dilution tubes. Increasing the number of samples for each dilution may remedy this defect in the method but the loss of media, time and space due to unwieldiness may be far greater than is profited by a gain in accuracy when counting high numbers of organisms.

In spite of the toxicity of citrate ricinoleate agar of a 9.2 per cent reduction of Escherichia organisms and a 30 per cent reduction of

TABLE 3

Examination of water from Raritan River, N. J.

SAMPLE NUMBER	SOURCE	COLIFORM COUNT PER 1 CC. WATER	
		Lactose broth	Citrate ricinoleate agar
76	North Branch	4.5	16
77	" "	2.5	4
78	" "	0.0	1
79	" "	1.5	0
80	" "	9.0	10
81	" "	25.0	10
82	South Branch	2.0	0
83	" "	2.5	9
84	" "	2.5	1
85	" "	1.0	2
86	" "	1.0	2
87	" "	4.0	2
88	" "	2.5	1
89	Manville	250.0	117
90	"	250.0	169
91	"	0.0	0
92	"	240.0	240
93	Weston	1.1	0
94	"	4.5	46
95	"	4.5	3
96	"	12.0	8
97	"	7.5	8
98	"	4.5	3
99	Green Brook	95.0	113
100	" "	15.0	85
101	" "	890.0	840
102	Bound Brook	25.0	31
103	" "	45.0	52
104	" "	950.0	1180
105	" "	35.0	30
106	Landing Bridge	20.0	26
107	" "	250.0	107
108	" "	13.0	40
109	" "	71.0	123
110	Johnson's Dock	250.0	140
111	" "	95.0	390
112	" "	0.0	0
113	Sayreville	95.0	130
114	"	45.0	70
115	"	250.0	190
116	Victory Bridge	25.0	34
117	" "	7.5	31
118	" "	4.5	5
119	" "	45.0	100
120	" "	45.0	100
121	Sewage effluent	20	28
122	Sewage effluent	15	11
Average count		88	96.1

Aerobacter organisms reported in the first part of this paper, the average productivity of this medium in the examination of potable and river waters was found to be 30 per cent and 9 per cent greater

TABLE 4

Efficiency correlation of lactose broth with citrate ricinoleate agar in examination of river water

DESCRIPTION OF SAMPLE	NUMBER OF SAMPLES
Total examined	47
Coliform negative	2
Coliform positive in either media	45
Positive in lactose broth	44
Positive in CR[1] agar	42
Positive in CR agar	93 per cent
Positive in lactose broth	97 per cent
Higher productivity of lactose broth to agar	4 per cent
Positive in both media	41
Positive in LB,[2] negative in CR agar	3
Negative in LB, positive in CR agar	1
Higher count in lactose broth than in CR agar	15
Higher count in CR agar than in LB	25
Total positive showing higher count in LB than in CR agar	37 per cent
Total positive showing higher count in CR agar than in lactose broth	63 per cent

[1] Citrate ricinoleate.

[2] Lactose broth.

TABLE 5

Examination of sewage

SAMPLE NUMBER	COLIFORM COUNT PER CC.	
	Lactose broth	Citrate ricinoleate agar
1	60,000,000	46,000,000
2	100,000,000	95,000,000
3	15,000,000	12,000,000
4	1,300,000	1,500,000
5	30,000,000	25,000,000
6	1,100,000	900,000
Average	34,600,000	31,700,000

in actual numbers of coliforms than lactose broth but 9 per cent lower in examination of sewage. Contributing to the somewhat higher productivity of citrate ricinoleate agar in the examination of potable waters may be the presence of slow gas-forming coliforms

many of which are not detected in lactose broth at the end of the 48-hour test. It must be remembered that identification of Escherichia and Aerobacter organisms on citrate ricinoleate agar depends on the ability of these organisms only to produce acid from lactose in the presence of 0.1 per cent sodium ricinoleate. Gas formation with its inevitable false tests in lactose broth is not a criterion of the coliform group on citrate ricinoleate agar.

Due to the limitation upon the number of water samples one investigator can collect and analyze comparatively over a reasonable length of time, the scope of this study is quite limited when compared to other coöperative coliform studies which have involved thousands of samples. Nevertheless, the results obtained here indicate the reliability of citrate ricinoleate agar as a rapid routine test for the coliform analysis of potable and polluted waters. Furthermore, the much needed separation of Escherichia count from the Aerobacter count now possible with citrate ricinoleate agar is of especial interest because of its utilization in the analysis of shellfish in which major importance is attached to the Escherichia coli count.

The advantages which citrate ricinoleate agar appears to possess over standard lactose test may be enumerated as follows: 1. An equal or slightly higher productivity of citrate ricinoleate agar over the standard lactose test. 2. Rapid observation and counting of coliforms possible with citrate ricinoleate agar with a completed count in 24 hours. The time required for the standard lactose test may vary from 2 to 4 days. 3. Elimination of false test and synergistic organisms by citrate ricinoleate agar and the elimination of the confirmation of false positive presumptive tests which characterize the standard lactose test. 4. Ability to make a differential count between Escherichia and Aerobacter organisms. 5. Ability of Eberthella, Salmonella and Shigella to grow on citrate ricinoleate agar with the inclusion of these pathogens among the coliforms in the classification. The standard lactose test fails to detect these organisms. 6. Higher counting accuracy is obtainable by a plating method against a dilution method per tube and plate of media. 7. Use of citrate ricinoleate agar enables a reduction in amount of media, time and space necessary for routine analysis of water.

REFERENCES

1. Littman, M. L. and Stark, C. N. Citrate-Ricinoleate Agar for the Detection of Escherichia, Aerobacter and Proteolytic Gram-Negative Rods in Milk. Jour. Bact. **34:** 348 (1937).
2. *Standard Methods of Water Analysis*, Eighth Edition, *Am. Pub. Health Assn.*, New York, (1936).

PROPERTIES AND DETERMINATION OF METHANE IN GROUND WATERS

By T. E. Larson

Recent developments (1) in the recognition of the hazards accompanying the presence of methane in ground waters have warranted further investigations concerning the dangerous or limiting permissible concentration. A general definite value cannot be given, but several limiting statements can be reported, and each particular situation must be considered accordingly. Any place where methane-bearing water comes in contact with air is a potential point of interest.

As a gas, methane is colorless, odorless, and tasteless. Dissolved in water it is chemically unreactive and cannot be distinguished from air except for the fact that it burns or explodes on mixing with air after release. Sometimes it is accompanied by the presence of a trace of hydrogen sulfide.

The solubility in water is 5.4 cubic feet (as gas) per 1,000 gallons of water when in contact with an atmosphere of 100 per cent methane at atmospheric pressure. If the gas dissolved in the water is 100 per cent methane and the total gas content is greater than 5.4 cu. ft. per 1,000 gal., the reduction in pressure on bringing ground water to the atmosphere will release the supersaturated gas as small bubbles within the body of the water.

Should dissolved nitrogen also be present, as is frequently the case, or less prominently free carbon dioxide, the value for supersaturation is decreased. Since nitrogen solubility is 2.5 cu. ft per 1,000 gal., supersaturation when the gas is 100 per cent nitrogen will occur on pressure reduction to atmospheric, if more than this value is present. In figure 1, curve CF shows saturation concentrations for mixtures of nitrogen and methane dissolved in water under atmospheric pressure.

Release to atmosphere in cases of supersaturation is not dependent

A paper presented at the Illinois Section meeting, Decatur, Ill., April 5, 1938, by Dr. T. E. Larson, Chemist, State Water Survey, Urbana, Ill.

on the relatively slow diffusional processes through the liquid and liquid film and the extent of surface exposed to air, but upon barometric pressure. A drop in barometric pressure can make an undersaturated solution supersaturated and eliminate the slow diffusion process for release by releasing the gas actually within the liquid thus permitting it to rise rapidly to the surface.

If the dissolved nitrogen is sufficiently high in concentration relative to methane, it is possible that no dilution of the released supersaturated gas with air can produce an explosive or inflammable mixture. In the presence of excess nitrogen, oxygen must be present at least to the extent of 12.8 per cent and no methane-nitrogen mixture containing less than 14.3 per cent methane can form an explosive or inflammable mixture on dilution with air (2).

No supersaturated gas solution can form a methane-nitrogen vapor of 14.3 per cent methane if the dissolved methane percentage is less than that of equation 1 represented in figure 1 by the curve BE.

$$\begin{array}{l}\text{Per cent of methane} \\ \quad\text{in dissolved gas} = \dfrac{72}{\text{cu. ft. } (CH_4 + N_2)/1000 \text{ gal. water}}\end{array} \qquad (1)$$

Therefore, it is seen that definitely hazardous waters may be classified as those having greater total gas content than the values indicated by line EF in figure 1 and greater methane percentage than the values indicated by the line BE. However, it must be recognized that these saturation values are somewhat ambiguous in that they are true only within the body of the liquid. Simultaneous with the rapid expulsion of gas above saturation, gas is also released relatively slowly by actual diffusion. This continues until the methane concentration in the air is in equilibrium with that dissolved in the water.

On making the possible assumptions of no ventilation and the presence of a potential source of ignition, the dangerous concentration in the water must be limited by the percentage concentration it can produce in the air. This percentage is taken as 5 per cent. Any higher percentage will be explosive or inflammable on ignition. When it is realized that only one volume in every twenty volumes of air need be replaced by methane to form an explosive or 5 per cent mixture, it can be seen that little methane will have to be released from the water to form a dangerous hazard.

The ultimate limit for methane concentration in water is 0.27 cu. ft. per 1,000 gal. as obtained from the equilibrium relation,

$$\% = 272\ Q/(14.7 + P) \qquad (2)$$

where % = the concentration in the air, Q = cu.ft. methane/1,000 gal., P = the pressure in lb./sq.in. above atmospheric, and a tem-

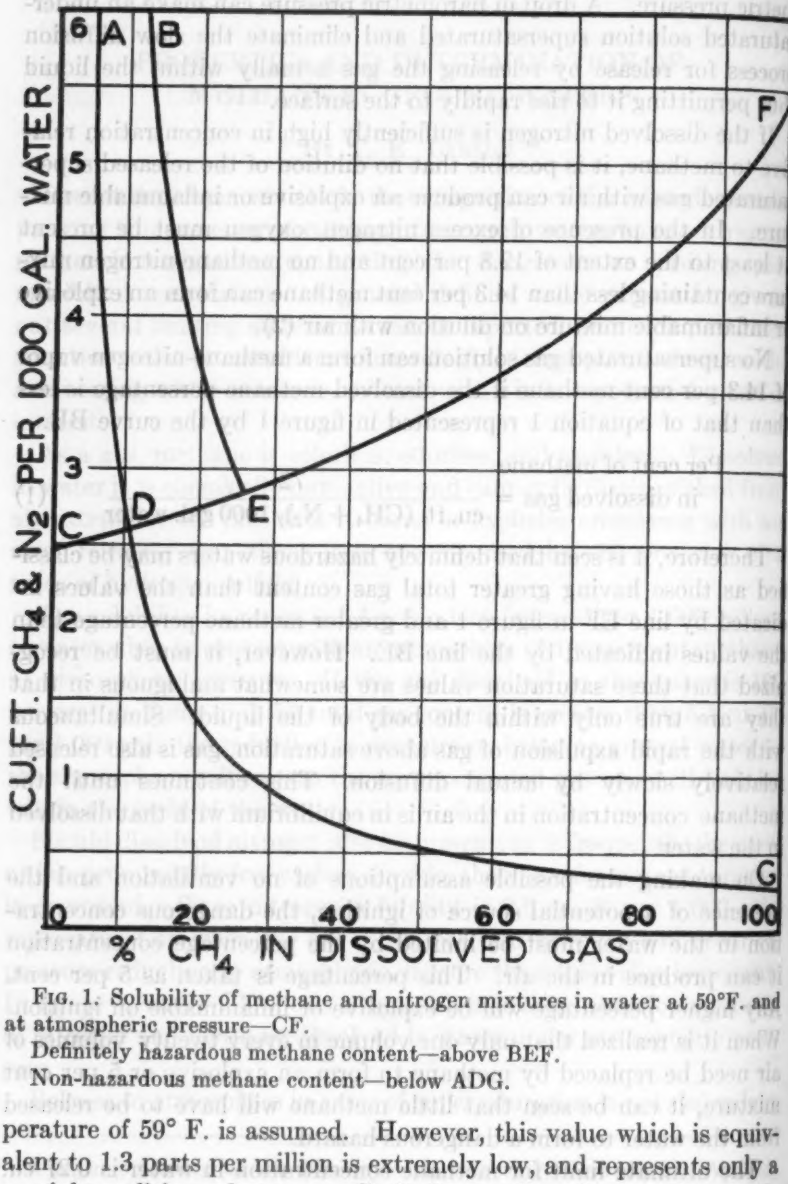


Fig. 1. Solubility of methane and nitrogen mixtures in water at 59°F. and at atmospheric pressure—CF.

Definitely hazardous methane content—above BEF.

Non-hazardous methane content—below ADG.

perature of 59° F. is assumed. However, this value which is equivalent to 1.3 parts per million is extremely low, and represents only a special condition where an unlimited quantity of aerated water is

sprayed through an unventilated chamber of air. This condition may be considered impossible of attainment so this value need not be labeled as the necessary maximum permissible concentration. The limiting dangerous concentration can be between 0.27 and 5.4 cu.ft. per 1,000 gal.

It can be seen that the quantity of water exposed to the air is a factor to be considered. Equation (3) has been developed to predict the minimum quantity of water which can produce an explosive methane-air mixture if as much methane leaves the water as is possible and no air is dissolved in the water, or all depleted air is constantly replaced. For this to happen, the water must be sprayed and become intimately mixed with the air. It is realized that this condition is theoretical and never happens in practice, but the values obtained do represent the minimum quantity of water necessary and are important when accepted from this standpoint.

$$H_2O = \frac{5371}{(5.4 - Q)^2} \log_e \frac{.95\,Q}{(Q - .27)} - \frac{52.66}{(5.4 - Q)} \qquad (3)$$

where Q = cu.ft. methane per 1,000 gal. water, and H_2O = gal. of water per cu.ft. of air.

Figure 2 shows the minimum quantity of water in gallons which can produce an explosive or 5 per cent mixture of methane in one cubic foot of air.

Although the transfer of methane above saturation from water to air is relatively instantaneous, calculations on the rate of transfer by diffusion cannot be made without knowledge of film coefficients, K_L, and the extent of surface, S, in contact with air. Since the extent of surface varies with each particular situation, such calculations would be superfluous. It is to be noted that the rate of transfer is directly proportional to the exposed water surface.

The particular points of interest should be those where much water comes in contact with a small quantity of air. These points are at the well, between casing and the discharged pipe, in storage reservoirs and in small unventilated rooms with constantly flowing water. The air cushion in the pressure tank is a likely place, but here the total pressure is effective and the per cent methane that may be built up in the air cushion is inversely proportional to the absolute pressure, (equation 2). To state it in another way, the greater the pressure the lower will be the possible methane percentage in the air.

Schools are of particular interest in respect to methane hazards

because of the periodic concentration of people using water. Several have had serious explosions due to methane in the water and others have been found where explosions could take place. Water works small and large are of interest because the large volumes of water handled make such a low concentration of methane in the water potentially hazardous. New buildings will be responsible for more serious situations than old because of the better construction in general, with less cracks and better fitting doors and windows preventing any degree of ventilation.

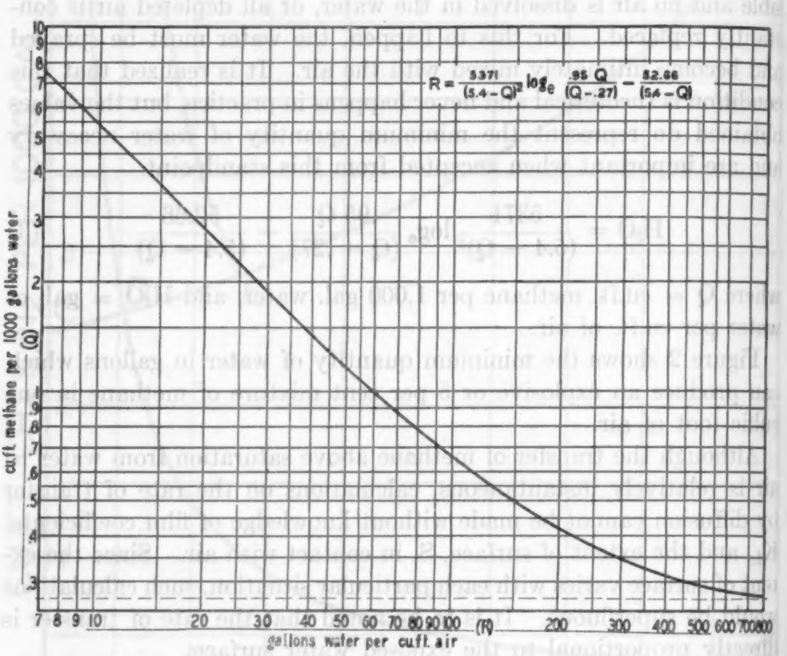


FIG. 2. Minimum quantities of water which would be necessary to produce an explosive methane-air mixture.

Means of reducing the possibility of hazards consist of thorough ventilation at all times and elimination of sources of ignition. Complete prevention consists of aerating the water directly after leaving the well. The well casing should be sealed air tight and if it is desirable to vent the casing, the vent should extend outside the building and out of reach by hand. If it is desirable to vent a pressure tank to release gas pressure which sometimes builds up in the air cushion, this vent should also extend outside the building. Care

should be taken in draining and cleaning the pressure tank to release all the gas outside the building.

Tests on various aeration units indicate methane removal to at least .6 cu.ft. per 1,000 gal. At this concentration no more than 4.7 per cent methane can be attained in a pressure tank operating at a minimum gage pressure of 20 lb. per sq.in. and a minimum of 115 gal. of water will have to come in contact with one cubic foot of air to obtain an explosive mixture. Several small, outdoor, three-tray, coke aerators with a water-tight roof and twenty-four mesh, screen walls such as those suggested by the Illinois State Department of Public Health are in service and giving very satisfactory results.

From the data presented, an approach can be made toward classifying a water as dangerous or safe. From analysis a water may be classed in any of four classes as charted in figure 1. Complete safety is assured if the methane content is below 0.27 cu.ft. per 1,000 gal., as indicated by the area below curve ADG. A possible hazard is indicated in the area above this curve. A definite hazard is present if the water is supersaturated with gas of sufficiently high methane percentage as indicated by the area bounded by the curves BE and EF. It can also be seen that under certain conditions waters of gas content placing them in the area bounded by AD, DE, and BE must be classed with the other possible cases included in the DFGD area where the release of gas is controlled entirely by diffusion and the extent of air-water surface.

Naturally the higher the values, either total gas or per cent methane, the greater the potential hazard. Further considerations and limitations may be obtained from figure 2 which predicts the minimum quantity of water necessary to make a definite volume of air potentially explosive. The extent of air-water interface is a variable but potent factor and can make a possible hazardous water more dangerous than a definitely hazardous water.

Consideration has been given on the quantity of gas present, the quality of this gas, the rate of escape from water, and the relative quantity of water necessary per quantity of air. The effect of one other factor, pressure, is indicated by equation 2. All calculations involve the assumption of constant temperature of 59°F. It is realized that a twenty-degree rise in temperature will make an appreciable difference in results, but for the border-line cases where this is effective other factors such as the minimum quantity of water necessary, and the extent of surface are of greater importance.

The question of a dangerous water remains to a certain extent very much a question of possible and probable, but it is certain that thorough ventilation should be employed in either case. Aeration of the water is necessary in waters classed as definitely hazardous and very desirable in cases of possibly hazardous methane concentration.

REFERENCES

1. BUSWELL, A. M., AND LARSON, T. E. Methane in Ground Waters. Jour. A. W. W. A., 29: 1978 (1937).
2. COWARD, H. F., AND JONES, G. W. Bureau of Mines Bull. 279, p. 45, 1930.

Discussion by H. A. SPAFFORD.* I wish to emphasize the two general ways in which methane gas may issue from water to create serious gas explosions.

The gas may issue directly from the water while it is still in the well, pass up through the well casing and, if the casing is not tightly sealed, enter the pumping station or well pit, where it may mix with air and an explosive mixture result. It is, therefore, apparent that well casing tops should be tightly sealed not only as a sanitary protection but also to eliminate possibility of methane gas explosions. It is also important that the well casing be vented with a pipe of sufficient size extending outside any enclosed structure. The lack of such a satisfactory vent resulted in the serious gas explosion last fall at the Normal, Illinois public water supply, where the well casing was properly sealed but was vented into the pump room. Methane gas issued from the vent, resulting in an explosive mixture, and a disastrous explosion.

We have found from our experience on sewage installations that vent pipes should be a minimum of about 2½ inches in diameter where they extend outside a building and are subject to freezing. If a size smaller than 2½ inches is used, there is danger of moisture condensing and freezing on the inside of the pipe and closing the opening.

Proper well top sealing and venting as outlined above will satisfactorily eliminate the possibility of pump house and well pit explosions but the water pumped from the well will still contain dissolved methane. If the water is placed in enclosed storage structures such as a collecting reservoir, methane gas may evolve from the

* Assistant Engineer, Illinois State Dept. Health, Springfield, Ill.

water and result in an explosive mixture between the water surface and the reservoir roof. This condition resulted in the destruction of the concrete collecting reservoir at the Clinton, Illinois public water supply early last fall. In order to protect against this type of explosion it is necessary to provide some form of treatment to remove the methane gas from the water before it is placed in storage structures.

From the best information now available it appears that a multiple coke tray type of aerator, over-designed as to capacity, will satisfactorily remove methane to a point where there will be little likelihood of an explosive gas mixture resulting after such treatment. The aerator should be designed preferably on a basis of about 1 gallon per square foot per minute of total tray area and not more than 2 gallons per square foot per minute. The aerator should be located outdoors where the evolved methane may be dissipated in the atmosphere and of course the aerator should be screened with 24 mesh noncorrosive screen to minimize contamination. After aeration, water should be collected in a gravity storage reservoir or sump, from which it may be picked up by a high service pump and placed under pressure for distribution.

A bulletin has been issued jointly by the Illinois Department of Mines and Minerals, Illinois State Water Survey, and the Illinois Department of Public Health. This bulletin describes briefly the methane gas problem and the above suggested method of eliminating explosion hazards. These bulletins may be obtained without charge upon request to the Illinois Department of Public Health.

In conclusion, I wish to commend Dr. Larson upon this splendid study of the methane gas problems at water supplies. Dr. Larson had to develop a method for determining the amount of methane gas in water. There was nothing previously in the literature on this problem and all of the analytical methods developed are entirely original.

MANGANESE AND IRON DEPOSITS ON SAND AND ANTHRAFILT FILTERS

By R. B. Adams

In 1910, the Pennsylvania Water Company located at Wilkinsburg, Pennsylvania completed the construction of a rapid sand filtration plant. The pumping station was located at Nadine, Pennsylvania, and the source of supply was the Allegheny River. Two large filter cribs had been built in the river bed some years previously. The supply to be used at the new plant consisted of part raw river water, part crib water, and some ground water. The exact proportions of these are not known.

After a few months operation of the new plant, manganese deposits were found on the gravel of the rapid sand filters, and from that time until 1930, many filters were rebuilt as a result of this troublesome element. During that period of twenty years, manganese never deposited on the sand to any serious extent.

The Allegheny River since 1910 has shown a marked decrease in alkalinity. Mr. Drake of Pittsburgh reports, "Due to industrial and mine wastes, the annual average alkalinity of the Allegheny River at Aspinwall, Pennsylvania has fallen from 24, in 1909, to 5 parts per million in 1929. Analyses have shown quite conclusively that the decreased alkalinity has been brought about by wastes from and below the Kiskiminetas River, which is approximately 22 miles above the Pittsburgh intake." These results do not imply that the river is constantly acid; normally it is an alkaline stream with sudden increases in acidity occurring usually in the summer or early fall months, and lasting about three months of each year. It has been found that manganese is usually present in greater amounts during the acid season.

Table 1 shows the annual average results of the raw water from the years 1931–37 inclusive. The alkalinity of our mixed waters

A paper presented at the Central States Section meeting at Wheeling, W. Va., August 18, 1938, by R. B. Adams, Chemist, Pennsylvania Water Company, Wilkinsburg, Pennsylvania.

is generally higher than that of the raw river water and it changes in proportion to the change in the river. For example, 1934 was our most acid year; the average alkalinity during 1934 of the raw water was 7.8, while that of the river water was 0.3 p.p.m. The difference or 7.5 p.p.m. was the amount of alkalinity obtained from the ground water.

The treatment used on the supply ever since 1910 has been alum and lime. Alum coagulates most efficiently on this water at a pH of 6.8 to 7.0, and the reaction is usually carried at this point. During acid periods, or when the pH of the raw water is below 5.5, no coagulant is necessary. Iron is ordinarily removed without difficulty as long as the pH in the mixing troughs and sedimentation basins is

TABLE 1

Raw water characteristics, 1931–37, in parts per million

YEAR	TUR.	COLOR	ALK.*	ACIDITY	pH	HARDNESS	Fe	Mn
1931	50	5.8	11.0	9.5	6.1	126	1.2	0.66
1932	42	6.6	12.0	8.2	6.2	128	1.8	0.62
1933	70	4.7	9.4	13.0	5.8	116	2.3	0.94
1934	48	3.2	7.8	15.0	5.7	119	3.3	1.1
1935	56	5.8	12.0	10.0	6.1	108	2.4	0.80
1936	48	5.8	12.0	10.0	6.1	112	2.4	0.91
1937	78	9.4	19.0	6.6	6.4	111	0.9	0.40

* Methyl orange alkalinity.

no lower than 6.8. Occasionally a pH of 7.4 is necessary for good removal.

Manganese, on the other hand, requires much higher alkalinities for precipitation than iron. It is necessary to carry 2 or 3 p.p.m. of caustic alkalinity to completely precipitate manganese unless an oxidizing reagent is used. Chlorine and potassium permanganate were tried in this connection, but were unsuccessful because of resultant tastes and odors. Manganese removal, with the regular alum and lime treatment as described above, is about 25 per cent, 15 per cent being removed in the sedimentation basins, and 10 per cent through the filters.

FILTERS

There are 16 one-million-gallon rapid sand filters: 10 of these are what we term shallow filters; the other 6, deep ones. All are equipped

with the Leopold underdrain system and are back-washed at a 24-inch rise per minute rate. The shallow filters contain about 11 in. of gravel, 20 in. of sand, and have a 17-inch freeboard. The deep filters contain about 30 in. of gravel and 25 in. of sand with a 25-inch freeboard. In 1930 the effective size of the sand in the shallow filters was about .55 mm. One of the deep filters contained a .55 mm. sand, while the remaining 5 deep filters contained a .45 mm. sand.

In 1935 we replaced one of the deep sand filters with anthrafilt. This unit was made up as follows:

10 in.—1-inch size gravel
4 in.—buckwheat coal (through $\frac{9}{16}$ and over $\frac{5}{16}$)
6 in.—rice coal (through $\frac{5}{16}$ and over $\frac{3}{16}$)
34 in.—anthrafilt

With increasing amounts of manganese in the raw water during the years 1933 and 1934, manganese deposits on the gravel of our filters became serious. Deposits of this element cemented the gravel to such an extent that wash water distribution was very poor.

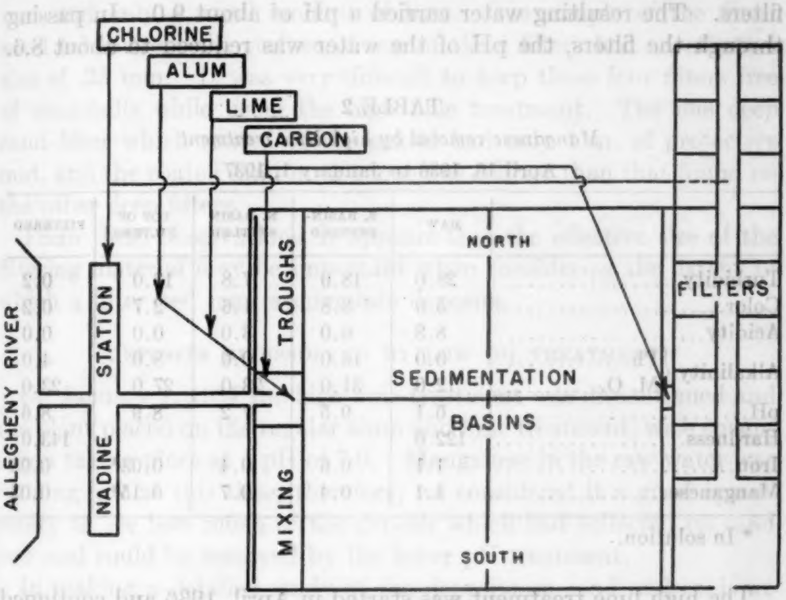
Our first procedure in loosening the gravel was to use a special longhandled rake which was pushed down into the gravel while the filter was being washed. The shallow layers reacted well to this procedure. Manganese still deposited on the shallow gravel layers, but hard spots could be kept loosened with the rake. The deep filters contained so much gravel that it was impossible to remove the hard spots by raking.

Our next step was to try a caustic soda treatment. The shallow filters were treated in place with a 1 per cent caustic soda solution. The caustic loosened the deposits from the gravel and set free large particles of manganese dioxide, which were washed from the bed. This treatment did an excellent job on the shallow filters; the gravel was loose and clean; the sand, which was coated with a reddish-brown deposit, was also well cleaned. The gravel in the deep filters was too badly coated to treat the entire layer in place, so the sand was removed and the gravel treated with a 1 per cent caustic soda solution. Using this procedure, the deep gravel layers were loosened but they could not be washed as clean as the shallow ones. After the sand was returned to the deep filters, and the units placed in operation, each filter wash would show a small amount of manganese dioxide scattered over the top of the sand. It was not being deposited but simply being washed from the gravel. Once a week for several months, this material was removed from the top of the sand.

By repeating the caustic soda treatment on both shallow and deep filters in place, once yearly, troublesome manganese deposits were reduced to a minimum.

A number of plants have reduced the coatings on their sand and gravel by using the caustic treatment. Our experiences indicate that it is much less effective if carbonates are present to any extent.

Priester has reported excellent results with chlorine in removing manganese deposits from both gravel and sand, when the deposit was free of carbonates. Jackson has used chlorine successfully in treating manganese and iron deposits on sand when excessive carbonates were present. Patrick has reported several instances where sulphur dioxide has been used in cleaning deposits of this nature. (See references at end of paper.)



PENNSYLVANIA WATER COMPANY—FLOW SHEET OF TREATMENT

MANGANESE REMOVAL BY HIGH LIME TREATMENT

In 1935 manganese was present in large amounts during hot weather, and complaints concerning manganese stains in the laundry and on bath room fixtures were numerous. In 1936 we decided to use a high lime treatment for a time to relieve this condition, and at

the same time to study the deposits which built up on the filter layers in the absence of recarbonation.

The high lime treatment to be studied is actually a split treatment and does involve some recarbonation. Referring to the flow sheet, the north sedimentation basin was treated as in the past with alum and lime, and maintained at a pH of 7.0 to 7.2. The south sedimentation basin was treated with lime alone in sufficient amounts to produce an OH alkalinity of 2 to 4 p.p.m. The pH at this point ran 9.4 to 9.6. The two differently treated waters passed across their respective basins and were mixed just before going to the filters. The resulting water carried a pH of about 9.0. In passing through the filters, the pH of the water was reduced to about 8.6.

TABLE 2

Manganese removal by high lime treatment

April 16, 1936 to January 1, 1937

	RAW	S. BASIN SETTLED	N. BASIN SETTLED	TOP OF FILTERS	FILTERED
Turbidity	28.0	18.0	7.8	12.0	0.2
Color	5.0	3.8	1.6	2.7	0.2
Acidity	8.8	0.0	3.0	0.0	0.0
Alkalinity Ph	0.0	16.0	0.0	8.0	4.0
Alkalinity M. O.	12.0	31.0	23.0	27.0	23.0
pH	6.1	9.5	7.2	8.9	8.6
Hardness	122.0				143.0
Iron	1.1	0.6	0.4	0.03*	0.08
Manganese	1.1	0.4	0.7	0.15*	0.03

* In solution.

The high lime treatment was started in April, 1936 and continued until January 1, 1937. Table 2 illustrates the type of raw water handled during the high treatment and shows results obtained. The high treatment removed 97.3 per cent of the manganese.

Under these given conditions, it is interesting to study the deposits which built up on the filter beds. The sand and coal showed signs of coating at the end of two months of high treatment. Manganese deposits were evident and laboratory tests indicated the presence of carbonates. At the end of three months, it was found that the filters were not coating in the same manner. All ten shallow filters, having the coarser sand, were coating evenly to the gravel, but not on the gravel. The deep anthrafilt filter and one of the deep sand

filters (filter 14 which contained a .55 mm. sand at start of high treatment) were also coating to the gravel. The other four deep sand filters were coating only about 3 to 4 in. down from the surface of the sand. Throughout the high treatment, the filters continued to accumulate deposits in the above manner.

Upon examining the four deep sand filters which coated to a depth of 3 to 4 in., we found only the fine particles of sand coated. In other words, the fine grains of sand coated first, remained on the surface of the filter, and acted somewhat as a protective mat for the filter. There was no manganese or iron depositing below this mat and only a small amount of lime. One sample of the finest sand taken from the surface of one of these filters had an effective size of .25 mm. It was very difficult to keep these four filters free of mud-balls while using the high lime treatment. The one deep sand filter which coated to the gravel had only 1 in. of protective mat, and the coated sand in this mat was coarser than that found on the other deep filters.

From these observations, it appears that the effective size of the filtering material may be important when considering the extent to which a filter bed may accumulate deposits.

DEPOSITS REDISSOLVED BY LOW pH TREATMENT

On January 1, 1937 the high lime treatment was discontinued and the plant placed on the regular alum and lime treatment, with coagulation taking place at a pH of 7.0. Manganese in the raw water was running low at this time, therefore, we considered it a good opportunity to see how much of the deposit which had collected on sand and coal could be removed by the lower pH treatment.

In making a detailed study of the deposits on sand and coal, we had originally planned to use one of the deep sand filters for a comparison; but due to the formation of a protective mat of fine coated sand on the deep units, and the resulting varying conditions, we decided to choose a shallow filter. As stated previously, all the shallow filters coated in a manner similar to the deep anthrafilt unit. All samples for this comparison were collected at a depth of 10 in. below the surface of the filter.

Chart A shows the total pounds of deposit which built up on a cubic foot of sand and anthrafilt throughout eight months of high lime treatment, and the extent to which these deposits redissolved upon returning to a pH treatment of 7.0. At the end of the high

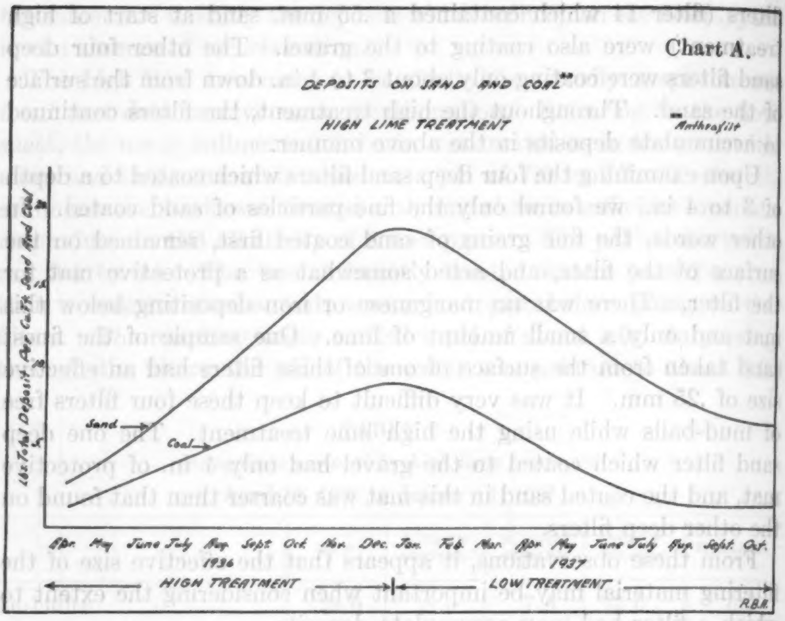


Chart A.

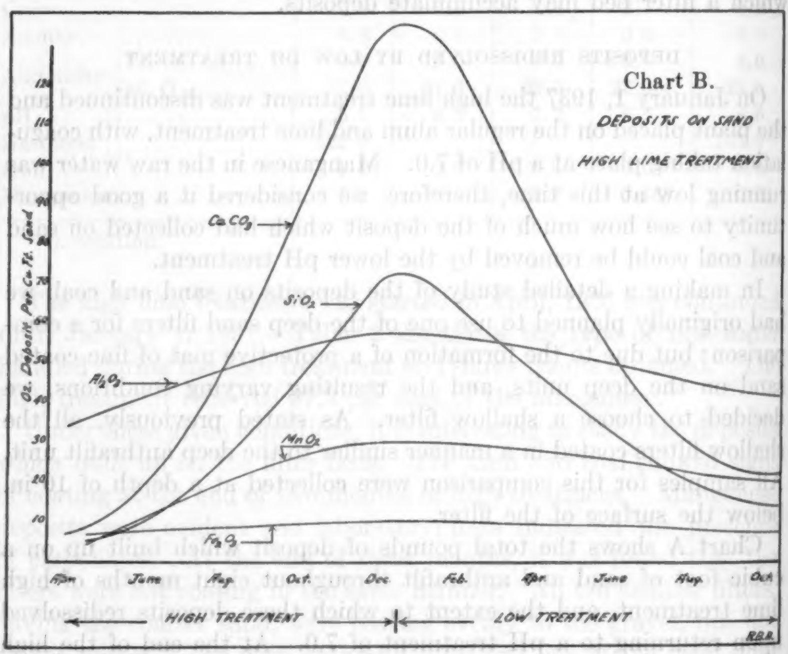


Chart B.

treatment, the sand had taken on 16 pounds of deposit per cubic foot, while the anthrafilt had taken on 7.75 lb. per cu. ft. The anthrafilt filter returned to its original condition, with the exception of a small percentage of manganese, in eight months time. The

TABLE 3

Deposit on sand

	TOTAL OUNCES DEPOSITED PER CU. FT. SAND	PER CENT OF REMOVAL BY LOW TREATMENT
Fe_2O_3	3.0	100.0
$CaCO_3$	128.0	88.3
SiO_2	67.5	81.6
Al_2O_3	23.5	70.2
MnO_2	26.0	38.4

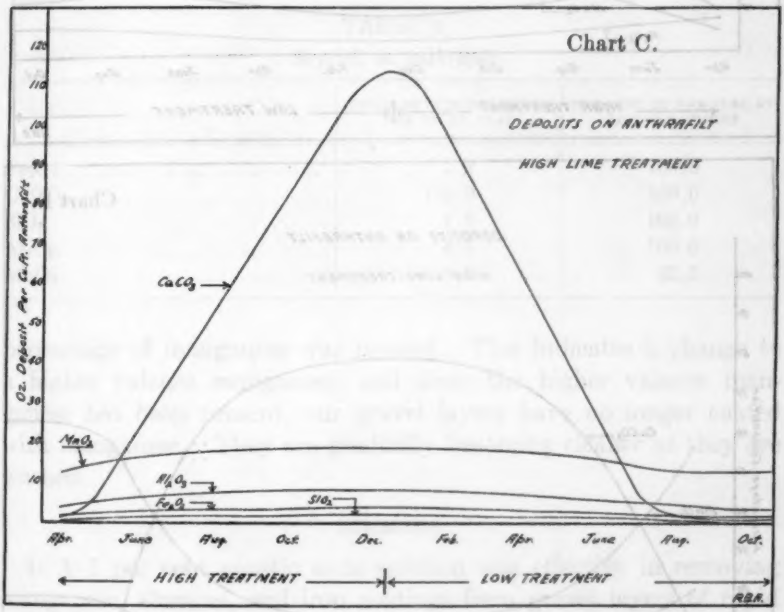


Chart C.

sand did not return to its original condition; only 76.5 per cent of the deposit was removed, and this required a ten-month period.

Chart B shows the ounces of $CaCO_3$, SiO_2, Al_2O_3, MnO_2, and Fe_2O_3 in the coating of a cubic foot of sand throughout the period of high and low treatment. Table 3 shows the total ounces of the various

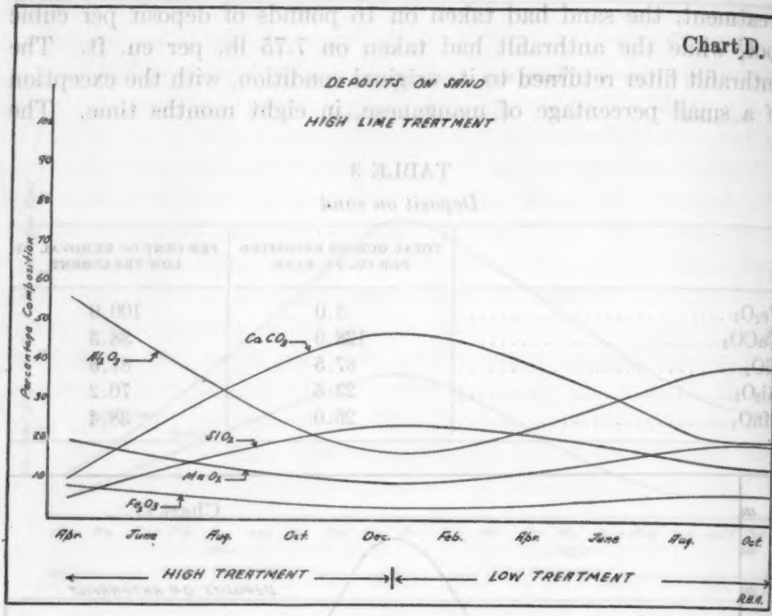


Chart D.
DEPOSITS ON SAND
HIGH LIME TREATMENT
Percentage Composition
CaCO$_3$
Al$_2$O$_3$
SiO$_2$
MnO$_2$
Fe$_2$O$_3$
Apr.
June
Aug.
Oct.
Dec.
Feb.
Apr.
June
Aug.
Oct.
HIGH TREATMENT
LOW TREATMENT
R.B.A.

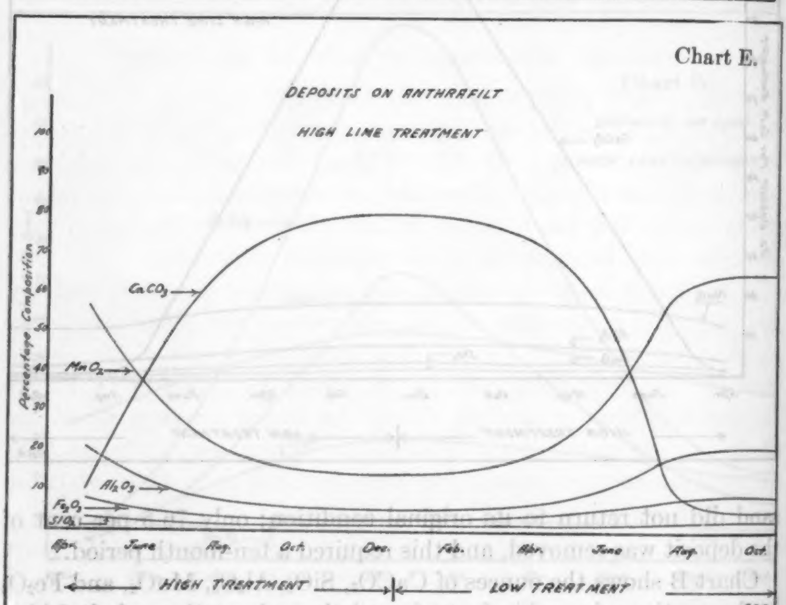


Chart E.
DEPOSITS ON ANTHRAFILT
HIGH LIME TREATMENT
Percentage Composition
CaCO$_3$
MnO$_2$
Al$_2$O$_3$
Fe$_2$O$_3$
SiO$_2$
Apr.
June
Aug.
Oct.
Dec.
Feb.
Apr.
June
Aug.
Oct.
HIGH TREATMENT
LOW TREATMENT
R.B.A.

compounds which deposited on a cubic foot of sand, and the percentage removal of these compounds by the low pH treatment.

Chart C shows the ounces of $CaCO_3$, SiO_2, Al_2O_3, MnO_2, and Fe_2O_3 in the coating of a cubic foot of anthrafilt throughout the period of high and low treatment. Table 4 shows the maximum ounces of the various compounds which deposited on a cubic foot of anthrafilt, and their percentage removal by the low pH treatment.

Charts D and E show the percentage composition of the coating on sand and anthrafilt respectively at any time during the high or low treatment. It is interesting to note that manganese in the sand coating, at the beginning of the high treatment, was present in the amount of 20 per cent and at the end of the redissolving period it still ran 20 per cent. The sand was originally reddish-brown in color; at the end of the redissolving period it was black, yet the same

TABLE 4

Deposit on anthrafilt

	TOTAL OUNCES DEPOSITED PER CU. FT. COAL	PER CENT OF REMOVAL BY LOW TREATMENT
Fe_2O_3	2.2	100.0
$CaCO_3$	113.0	100.0
SiO_2	1.6	100.0
Al_2O_3	4.7	100.0
MnO_2	6.5	92.5

percentage of manganese was present. This indicates a change to a higher valence manganese, and since the higher valence manganese has been present, our gravel layers have no longer coated with manganese. They are gradually becoming cleaner as they are washed.

SUMMARY

1. A 1 per cent caustic soda solution was effective in removing manganese, alumina, and iron coatings from gravel layers of rapid sand filters.

2. A high lime treatment for manganese removal built up deposits on both sand and anthrafilt.

3. The manner in which a sand filter coated with deposits depended upon the effective size of the sand.

4. At the end of eight months of high lime treatment, a sand filter

which had an effective size of .55 mm. at the beginning of the treatment, took on 16 lb. of deposit per cu. ft. Under similar conditions, a .70 mm. anthrafilt unit took on 7.75 lb. of deposit per cu. ft.

5. Seventy-six and one-half per cent of the deposit on sand was removed by the low treatment, while over 99 per cent of the deposit on anthrafilt was removed in the same manner.

6. Iron was low in both sand and anthrafilt coatings. Silica ran high in the sand coating but very low in the anthrafilt coating. Alumina and manganese both ran considerably higher in the sand coating than in the anthrafilt coating. Calcium carbonate deposited heavily on both sand and anthrafilt, the amount building up on anthrafilt being 88 per cent of that which collected on the sand.

7. Manganese dioxide coated on the sand grains is helpful in preventing similar deposits from cementing the gravel layers.

REFERENCES

1. Drake, C. F. Water Purification Problems in Mining and Manufacturing Districts. Jour. A. W. W. A. **23**: 1261 (1931).
2. Priester, Max U. Reconditioning Filters. W.W. and Sew. **81**: 48 (1934).
3. Patrick, J. G. Cleaning Filters With Sulphur Dioxide. W.W. and Sew. **85**: 90 (1938).

PRIVATE FIRE LINES IN NEW YORK STATE

By Wallace T. Miller

Ever since private fire lines were first installed, there has existed a continuous controversy concerning the advisability of a separate and distinct charge for the service furnished. Water works men substantiate their charges by the fact that water systems have two distinct functions, serving water for domestic and industrial consumption as well as for fire protection purposes, and they believe that these services should be charged for separately. They further state that private fire systems require installation, maintenance, and inspection of separate connections to the distribution system, much of which is outside of the complete control of the water department. They also say that the water department must stand ready continuously to supply full capacity of the lines if and when desired.

It may be further stated that the water department has provided a complete system of fire protection to benefit the entire public uniformly and those taxpayers requesting special and additional facilities should pay separately for such service. It is also a fact that private fire protection systems reduce insurance costs considerably and many water superintendents believe that the water department should share in savings which can only be accomplished through the use of the water supply system.

On the other hand are the arguments used by taxpayers, sprinkler manufacturers and others interested in the installation of fire protection systems. They contend that a private system represents no cost to the water department other than the normal inspection and supervision which might be covered by a nominal charge. The system replaces the demand on the public hydrants rather than increasing the demand, and if a separate charge is made, they feel the consumer is paying twice for the same service. Private systems are assumed to use less water in case of a fire and if any considerable charge is made,

---

A paper presented at the New York Section meeting at Poughkeepsie, N. Y., September 22, 1938, by Wallace T. Miller, Supt., Board of Water Com., Ossining, N. Y.

the contention is that a progressive citizen is being discouraged from benefiting the community.

There are many more arguments both for and against private fire line charges. The purpose of this paper is not, however, to substantiate the charges, but rather to show what represents common practice in New York state.

QUESTIONNAIRES SENT TO 103 WATER DEPARTMENTS

In order to accomplish this, a survey has been conducted covering practically all water departments in the state serving communities of over 5,000 population. Questionnaires were sent to 103 departments serving 130 communities. Replies were received from 96 departments serving 119 communities. The survey may therefore be assumed to have covered approximately 93 per cent of the departments of the state. None of the departments failing to reply represented a large city. The survey covered 85 municipally owned and 11 privately owned systems. Only three departments reported no private fire lines and approximately five more had somewhat incomplete returns. The results of the survey were as follows:

The first group of questions concerned the actual installation of connections from the street main to the building or property served. There were 48 departments which install connections with their own forces as far as the curb or property line as against 4 departments that make the tap only and 36 departments that allow the owner's agent to perform the entire work. The complete cost of this work is chargeable to the consumer in all cases but one, that community being a small up-state village.

The installation between the curb or property line and the building may be installed by the owner's agent in 65 communities, whereas 18 departments require installation by their own forces and 5 will permit installation by either agency. In every case the cost of this portion of the work is chargeable to the consumer.

The next group of questions concerned fire line meters. Twenty-five departments reported some type of meter required, 21 of which were more or less standard fire line meters and the other 4 represented the so called detector check, that is, an approved type of check valve with a small meter on a bypass. Eleven water departments assume the original cost of the meter, whereas 14 charge the entire cost to the consumer. Actual installation of the meter is performed by the department forces in 15 cases and the additional 10 depart-

ments allow the meter to be installed by a private plumber. In the majority of cases, however, the installation cost is chargeable to the consumer, only 8 departments out of 25 assuming this cost.

A separate annual meter charge is applied by only 3 of the 11 departments that do not require the consumer to pay the original cost of the meter. Two of these represent the cost and maintenance of the meter over its estimated life and the third is imposed by a department having an individual sprinkler head charge and requiring meterage only where hose outlets are desired.

ANNUAL CHARGE IS MOST COMMON METHOD

The most common method of charging for private fire protection is an annual charge based on the size of the connection and covering all phases of the service rendered. A total of 35 departments have some method of charging, 30 of which set a flat annual rate, 4 having an individual sprinkler head rate and one having a combination of both. There are 57 departments which have no charge whatever.

Four-inch fire lines vary in annual charges from $1 minimum to $250 maximum per year and the average of those having charges is $51 per year. Six-inch lines range from $1 minimum to $500 maximum and the average is $85. Eight-inch lines vary from $1 to $800 with an average of $147. Of the four departments having an individual sprinkler head rate and no other annual charge, the minimum is 2½ cents per head per year and the maximum is 10 cents with an average of 5.6 cents. Only one department in the state had an additional charge for hose outlets.

Charges for private hydrants apply in 16 departments and range from $10.00 to $75.00 per year with an average of $42.50 per hydrant. No differentiation was made as to whether these hydrants served private estates or were in conjunction with industrial or sprinkler lines, it being assumed that the rates would likely be the same.

Further information was requested concerning actual water consumed through private fire lines. Seven departments of the 25 with metered lines charge for water used for fighting fires and a total of 12, or approximately 50 per cent, charge for water consumed for other purposes, such as testing. One-half of the fire line meters are read monthly and the remainder are read quarterly.

Information was also obtained concerning the interval between inspections on private fire lines. Forty departments have regular inspection periods varying from one to twelve months and eight de-

partments inspect at irregular periods in order that their inspection may not be anticipated. There are, however, 41 departments in the state that apparently do not inspect their private fire lines, even though they may not be metered.

There are several minor but rather unusual features in connection with the requirements of some departments that appear worthy of mention. For instance, two cities in one section of the state report that they require the posting of a $1,500 bond to guard against unauthorized use of the system. Another city does not establish an annual charge provided the consumption of water by the user exceeds $100 per year. One city sets a flat rate on installation cost which includes a rather large profit and considers that this will cover cost of service for some indefinite period. Still another city is reported to keep the total private fire line revenue at a constant figure by reducing the charges to old consumers when new consumers are added.

One department formerly providing fire protection and industrial use from a single metered line has recently established a new class of service covering independent unmetered fire lines. Rates for the new service have been somewhat modified and requirements adopted providing for a connection around the valve so that a meter may be installed at intervals to test for leakage.

The fact that the subject of charges on fire lines is of great interest to water works superintendents is evidenced by the statement from several that their departments do not have any charges but that the superintendents feel that there should be a distinct charge. On the other hand are the statements of a lesser number of superintendents that they feel their sprinkler lines are an asset to the department rather than a liability, and therefore should not be charged.

Many municipal water departments have some method of applying a general fire protection charge over the entire property served. Several assess all real property in amounts varying from 75 cents to $2 per $1,000 valuation. Another department applies a frontage charge of 5 cents per front foot per year while still another has a fire protection charge based on the size of the street main in the amount of ½ cent per inch diameter per front foot per year.

One cannot assemble this information without drawing some conclusions. The writer firmly believes that the cost of furnishing fire protection, both public and private, should be separated from the cost of furnishing water for industrial and domestic consumption. Most of our municipal departments, however, are completely unconcerned,

not only about the cost of service, but also in respect to establishing rates on an equitable basis.

Some of our municipal departments operate on a self-sustaining basis and still furnish free fire protection to the city as well as water for other municipal uses. In this case, all fire protection costs are being paid on the quarterly water bill. Many other departments, however, receive some portion of the general tax budget to cover these services and charges may therefore be on a somewhat more equitable basis. Surely we cannot intelligently establish a rate for private fire protection until we have first determined the total cost of fire protection in general.

It is also apparent that no two communities are exactly alike in respect to establishing private fire line rates. Circumstances vary in all cases and each city requires individual study to establish a charge which will adequately compensate for the service being rendered.

It is also unfortunate to find that approximately 46 per cent of the departments in the state allow unmetered lines to remain in service year after year without any special or concentrated inspection. It is, of course, a fact that these represent the smaller communities and complete facilities may not always be available. However, the most casual inspection might prove of inestimable value, and future installations might be provided with a connection around the valve to apply a meter test, as mentioned previously, to simplify future inspections.

Editor's Note. The overall cost to industries of protection against loss from fire is a composite of many factors which are interdependent to such a degree that they should be studied together in any investigation of operating and maintenance expenses.

Fire protection may be divided roughly into two parts:

1. *Physical Protection*, dependent, among other factors, upon the construction and location of the buildings; the existence of proper zoning codes and building codes; neighborhood conflagration hazards; the character and handling of raw materials, inventories and wastes; the adequacy of public fire protection equipment and personnel; and the presence or absence of adequate supplementary private fire-fighting facilities. Water supply is important but by no means the only element to be considered in rating physical protection. Too often it is improperly weighted.

2. *Financial Protection,* afforded by various forms of insurance, the aggregate principal amount of which should, in general, be determined by each industrialist for his particular business and plant.

The industrialist should therefore, if possible, ascertain the relative values and costs of the two types of protection against loss of property and production and the most advantageous or economical combination of the two. He probably will think first in terms of premiums. The premiums paid for insurance should, on the average, be scaled down as physical protection approaches an ideal. There obviously must be a minimum premium, based upon average risks.

Rating bureaus charged with the duty of establishing an average risk rating for each of many areas or municipalities are faced with constantly changing physical conditions. New developments in building materials, fuels, chemicals; changes in building and plumbing codes; improvements in the art of fire prevention; increased efficiency in personnel and apparatus of fire departments; changing volumes and pressures of water supply available for preventing or extinguishing fires all have become so involved that it is not surprising to find recently that some municipalities were being rated on data gathered as far back as 1919. Yardsticks used in computing individual deficiencies and credits to be applied to base ratings become, for the same reasons, obsolescent in a few years and must be reviewed.

The average industrialist is not in possession of all of the information required for a decision as to the relative values of the several protections against fire losses for which he pays. He naturally wonders whether he is getting his money's worth. Any impartial investigation undertaken in his behalf must include not only a study of the efficiency and costs of the physical protections he receives, but also an inquiry into the local insurance rating, the schedules of penalties and credits and the restrictions and requirements contained in the policies for which he is asked to pay a certain premium. Except in the case of isolated plants, normal public fire protection is generally available to all industries whereas by no means all of the industries use or require private fire service. All industries and the general public pay the costs of public fire protection through taxes. There does not appear to be a sound reason why taxes should include the costs of rendering a separate and specific service which is required for special private protection in plants which for various reasons may

demand it. Keeping the broad aspects of the question in mind it would appear to be somewhat futile to concentrate any inquiry within but one aspect of one element of the overall costs of fire protection in an effort to further the sale and installation of sprinklers or other private fire-fighting equipment.

If an inquiry is made into private fire protection its scope should include, among other matters:

1. The present costs of sprinkler installations as bid under contract, also the cost of steel pipe, fittings, valves and heads.
2. The placing and serviceability of fire hose and its present cost.
3. The extent of use and adequacy of chemical extinguishers and their costs.
4. The present efficiency of the older sprinkler installations and the regulation and sealing of inlet valves.
5. The restrictions and regulations of the rating bureaus and insurance companies upon inter-connections between private fire lines and industrial water supply pipes. This phase should include inquiry into the demand upon small plants that oversize supply pipes be installed in the face of assertions that but a few sprinkler heads will need to function to quench incipient fires and that the resultant demand upon a water system will be negligible.
6. The volume and pressure and emergency continuity of the water supply as a basis for determining whether supplemental high storage tanks and perhaps supplemental private sources of supply and pumping will be required. If such are considered necessary their costs should be studied.
7. The charges, if any, for private fire line connections to the water system.

If these be subject of a special study the following considerations are of particular interest. Such charges are nil, nominal or specifically higher as a matter of governmental policy. The determination is made in nearly all instances by municipal authorities or by state regulatory bodies. This applies also to water rates in general.

Municipal water departments may or may not keep accounts separate from the general municipal accounts. They frequently do not pay taxes and sometimes their fixed charges and operating deficits are passed along to the general tax levy. They are seldom under regulation by the states as to charges and seldom have any incentive to allocate costs to the several classifications of service rendered in the community. Since there is no such thing as "free"

service, all costs must be paid by customers or taxpayers and the municipality determines whether or not any group shall be favored at the expense of the others.

Private water purveyors are generally under state regulation and the degree to which their costs are broken down and met by specific charges in the rate structures is a matter of state policy and control. In general the commissions tend to go into considerable detail. Some states, such as Wisconsin, have lumped private fire protection costs in as part of the public fire protection, whereas the Indiana and New Jersey Commissioners, after a long and thorough study, adopted a policy of considering the real potential demand for private fire service through separate connections as a definite standby expense to water purveyors and introduced into their rate structures a charge comparable to other demand or standby charges for water connections based upon the size of such connections.

Mr. Miller's study of private fire protection service charges in New York State records a wide diversity of rates. No comment, with the information now available, can be made concerning the correctness of these charges. That some are low; that some are nonexistent does not prove anything. The probabilities are that the cities making these charges have given little or no thought to the value of the service rendered. Water departments have much to do by way of improving their rate structures, and the studies by Ludwig (Jour. A. W. W. A., **29**: 617 (1937)) and by Nixon (Jour. A. W. W. A., **29**: 1837 (1937)) contain much valuable information concerning the costs of and charges for fire protection service.

WHAT IS ADEQUATE HYDRANT DISTRIBUTION

By George W. Booth

The fundamental answer to this question is that there should be a sufficient number of hydrants within a reasonable distance of any building or group of buildings subject to fire to furnish supply to fire engines of such size and number as may be used to extinguish that fire or control its spread. That answer is, of course, not very useful without further explanation.

The Grading Schedule of the National Board of Fire Underwriters contains a table specifying the fire flow to be available in cities and towns of various populations; the quantities range from 1,000 gallons per minute for a town of 1,000 population to 12,000 gallons per minute for a city of 200,000 population, and provide for an additional supply for a second fire in still larger cities.

Fire department pumpers are expected to have a capacity equal to two-thirds the required fire flow. Assuming a city of 100,000 population, the required fire flow is 9,000 gallons per minute and the pumper capacity would be 6,000 gallons per minute. That amount of water can be delivered by 10 engines at the average rate of 600 gallons each; some will deliver more than others, depending on the power of the engine and the length of hose lines used which will determine the pressure to be carried at the pumper. None of the hose lines should be more than 600 feet long, otherwise the use of pressure required to overcome friction loss will result in decreased discharge from the pumping engines. For example, a hose line 1,000 feet long will require, for any given nozzle discharge a pressure about twice that for a 400-foot line, with correspondingly decreased available pump capacity.

It happens too frequently that, although the distribution system has sufficient carrying capacity to deliver the required fire flow, the number and distribution of hydrants are not such as to make the

---

A paper presented at the New York Section meeting, Poughkeepsie, N. Y., September 22, 1938, by George W. Booth, Ch.Eng., National Board of Fire Underwriters, 85 John St., New York, N. Y.

supply available to the best advantage. It is safe to say that in most of the larger cities hydrant distribution in the principal mercantile district is reasonably adequate. In many of the smaller cities and towns where more hydrants are needed, the addition of a comparatively few will make a material difference.

The more or less commonly used unit for measuring distribution is that of linear spacing, but that is misleading and the proper measuring unit is that of "area served" per hydrant. For any given area served, the size and shape of city blocks will have a material influence on the location of hydrants and their linear spacing. For example, in Portland, Oregon, the city blocks are approximately 200 feet square and one hydrant at each street intersection will provide a satisfactory distribution of one hydrant to each 40,000 square feet. In Salt Lake City, however, the blocks are approximately 750 feet square and a distribution of one hydrant for each 40,000 square feet will mean 14 hydrants for each block; that would require hydrants spaced at intervals of about 100 feet along the street, or to better advantage four hydrants at each street intersection and intermediate hydrants along the sides of the block.

Thus far we have considered mainly the congested value or principal mercantile districts of cities, where the conflagration hazard is usually the highest and the demands for water are the greatest. Most cities will have manufacturing districts in which the fire flow requirements will depend upon the construction of buildings, their heights, areas and degree of congestion. These requirements will usually range from 3,000 to 10,000 gallons per minute, and the same principles for hydrant distribution will apply as outlined above.

For residential sections the fire flow requirements will range from 500 to 4,000 gallons per minute, depending on the character of construction of the buildings and the space separating them from each other. Small dwellings of the cottage type will require the minimum, and close groupings of apartment or tenement houses will need the maximum. In some cases, especially suburban communities in the vicinity of large cities, there has come in the past few years a considerable development of apartment houses, rather closely exposing each other, on properties formerly occupied by single family dwellings with plenty of space between them. Water departments should be alert in providing more hydrants and very often larger mains to supply them, in the vicinity of such developments.

There are situations where a group of buildings is accessible for

fire fighting purposes from only one side; as for example when bordering on a waterfront, or having a railroad yard or other inaccessible space in the rear. There have been cases where the hydrants along the street in front of such properties have been so close to the fire that they could not be used; so, consideration should be given to providing hydrants as a second line of defense at locations farther away from the possible fire exposure.

The subject of hydrant distribution and discharges available involves an interesting study which has been undertaken on a large scale by a number of city water departments. A paper on "Methods of Making Flow Tests and Their Value to Water Works Engineers" was presented by the writer at the New York Section meeting in 1924 (Jour. A. W. W. A., **12**: 2; 157 (October, 1924)). Since that time a complete survey of the distribution system of Detroit was made by the water department, including flow tests of all hydrants on the system, opened in groups of four, with residual pressure observed on a fifth hydrant. The information thus obtained made it an easy matter to determine the locations and size of reinforcing mains necessary to provide adequate fire protection. After these mains were laid the tests were repeated and the results showed that the desired increase in carrying capacity had been obtained. A similar practice has been followed in New York City, and for years past in Chicago.

In numerous cities it has become the practice to mark hydrants in a distinctive way, thus to indicate the approximate amount of water available from them, either by painting the taps in different colors or by painting on the barrel of the hydrant the size of the main to which it is connected. Your Association has adopted a color scheme which is similar to those adopted by the New England and other water works associations. The procedure of making a study of the fire protection supply available is a worthy activity, and the distinctive marking of hydrants will be useful to the fire department in selecting those which are most likely to furnish adequate supply to pumping engines.

In conclusion, it is well to say that the matter of hydrant distribution is one that is best handled by considering the needs of different sections of the city individually, rather than by the application of hard and fast rules. The fire chief is often able to advise on locations where more hydrants are needed, even if to no further extent than to estimate the number of fire companies likely to be used at a serious fire in a given locality.

THE ACTION OF VALVES IN PIPES

By Professor R. W. Angus

Valves on pipe lines are so common that one usually gives them little attention outside the manual process of operating and maintaining them, and yet they offer certain problems in connection with discharges and pressures during closures that are of direct interest. In connection with certain studies being made during last winter, the author had occasion to make some experiments on the action of various types of valves. The author presents some of the results herewith.

In order to learn the characteristics of the valves, arrangements were made to test some of them in the Hydraulic Laboratory of the University of Toronto. A pipe line was connected to a large vertical tank with an overflow which maintained the head constant within a fraction of an inch. The valves were then connected successively in this line and the discharges for different openings were weighed; and, further, piezometers, reading directly in feet of water column, were set up on each side of the valve, so as to determine the pressure loss therein.

There were about 10 ft. of uniform, smooth, clean 3-inch pipe between the tank and the valve. For the downstream side two series of measurements were made, one series with a piece of pipe 6 in. long discharging into the atmosphere, and the other series with a longer pipe below the valve, and arranged to produce any desirable back pressure. These series, therefore, corresponded to the use of the valve at the end of a line and in the middle of it, respectively, but the measurements showed that, with a given drop of pressure across the valve, the discharge was practically the same in both cases.

Unfortunately, the largest size of valve available at the time was 3-inch, but three types of these were secured, viz., a gate valve, a

A paper presented at the Canadian Section meeting, March 23, 1938, Windsor, Canada, by Prof. R. W. Angus, Head of Department of Mechanical Engineering, University of Toronto, Toronto, Canada.

globe valve, and a cone valve, these being loaned respectively by the City of Toronto, by the Crane Company, and by R. N. Austin of Dominion Wheel and Foundries, Limited, Toronto. The one referred to as the cone valve had a rotating conical plug similar to a shut-off cock, but the hole in the plug was cylindrical and of the full bore of the pipe.

The gate valve was flanged, but the other two were screwed. After each was fastened on the pipe, the stem was very gradually turned from the closed position until the first drops of water began to come through, this being called the closed position. From this procedure, the number of turns or part turns required to open the valve to give free waterway was determined. The discharges have been plotted against per cent of gate opening, by which is meant the number of turns of the spindle required to open the gate or globe valve to its test position, divided by the number of turns required to give full area of waterway, this fraction being multiplied by 100. For the cone valve, of course, angles of rotation of the cone take the place of the turns in the other valves. These experiments were carried out for each gate position (a) by opening to this position, and (b) by closing to this position; but no material difference was found as between the two ways in which a given position was reached.

The discharge through each valve for a given fraction of opening is taken to vary as the square root of the drop in pressure across the valve. In order to make convenient comparisons, the results have been plotted for a one-foot drop; and for any other loss of head, for example, 4 ft., the discharge for the given opening would be the square root of 4 times, or twice that shown on fig. 1.

RELATION BETWEEN GATE OPENING AND DISCHARGE

The curves on fig. 1 show the relation between pipe velocity and gate opening for one foot loss of head. The globe valve has the largest discharge at the smaller openings, while the cone has the smallest, and at 50 per cent full opening the pipe velocity with one foot loss of head was 1.73 ft. per sec. for the cone valve, 2.38 ft. per sec. for the globe valve, and 4.11 ft. per sec. for the gate valve.

At 61 per cent full opening, the cone and globe valves have the same discharge, corresponding to a pipe velocity of 2.64 ft. per sec., while the gate valve gives 5.20 ft. per sec., but after this the globe valve discharge remains nearly constant and the cone valve increases very rapidly, crossing the gate valve at 82.5 per cent full opening, where

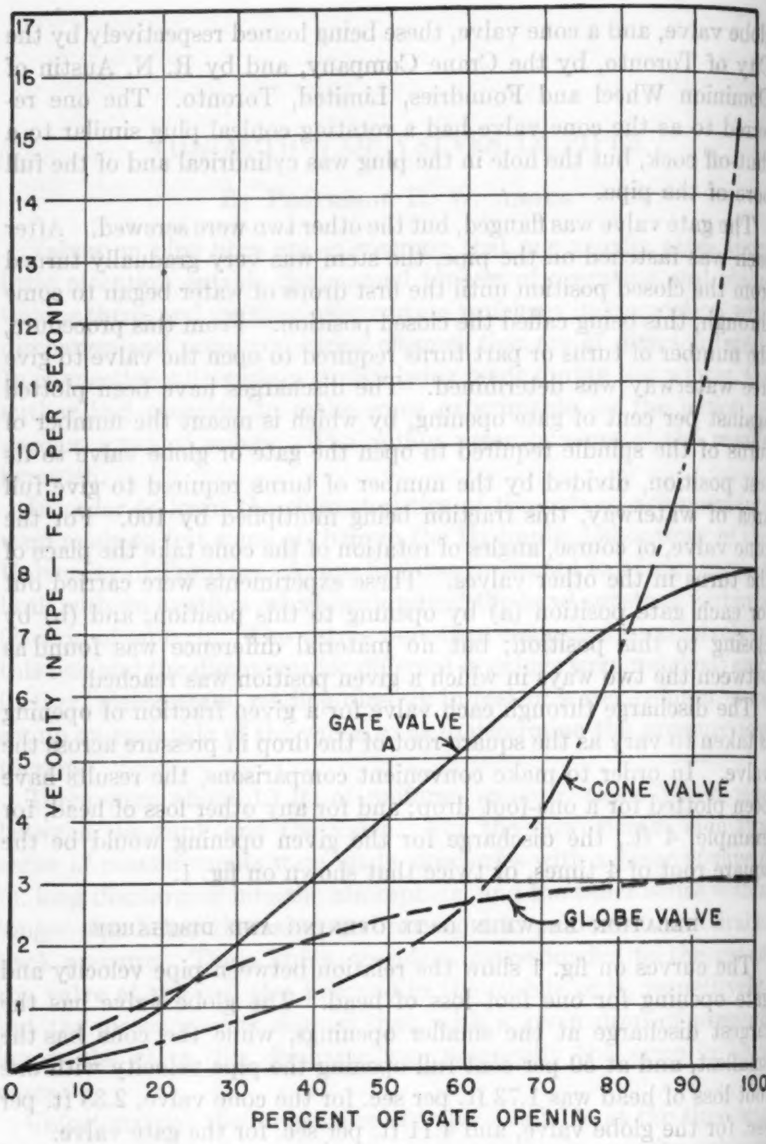


FIGURE 1. DISCHARGE THROUGH 3-INCH VALVES FOR ONE FOOT LOSS OF HEAD.

both give pipe velocities of 7.27 ft. per sec. At full gate opening the velocities are 25.30 ft. per sec. for the cone, 7.90 ft. per sec. for the gate, and only 3.04 ft. per sec. for the globe valve.

These tests show the flow that would result with 1 ft. loss of head for each valve when connected to a reservoir with a very short pipe. As the cone valve has little more resistance when wide open than the same length of pipe, the maximum discharge through it is much the largest, but its resistance increases very quickly when closure begins and its discharge therefore drops rapidly at first, but later on it changes very slowly. The type of discharge curve shown for the cone valve is desirable, as will be seen later, in avoiding danger from high pressures when shutting the valve in a long pipe line. On the other hand, both the globe and the gate valves show a very gradual reduction of discharge at first, leaving the velocity relatively high when the valve is well toward the closed position, a condition which is not desirable.

VARIATION OF DISCHARGE WITH CLOSURE

The curves of fig. 1 show that the statement frequently made, that the first part of the closure has little effect, is entirely wrong in some of the valves and not accurate in the others where the pipe is very short. For example, with the cone valve, closing the valve 10 per cent from full opening reduces the discharge very greatly, and a 20 per cent closure from full gate causes a reduction to 27 per cent of the original discharge, with the same loss of head in the valve in each case.

The gate valve is not so marked in its effect at first; 10 per cent reduction in gate opening cuts off only about 3 per cent of the discharge, and 20 per cent decrease in opening reduces the discharge by only 11 per cent, but after that the reduction in discharge is nearly proportional to the reduction in gate opening, down to about 20 per cent of the latter. It should be noted that when at the nearly closed position, both the cone valve and the gate valve reduce the discharge very gradually, the former more slowly than the latter.

The globe valve is different from the others and when the valve is moved from full open position to half open position, the discharge falls only 19 per cent. This valve further has the somewhat unfortunate characteristic of reducing the discharge rapidly as the valve is nearly closed.

The above statements apply to the valves themselves and not to

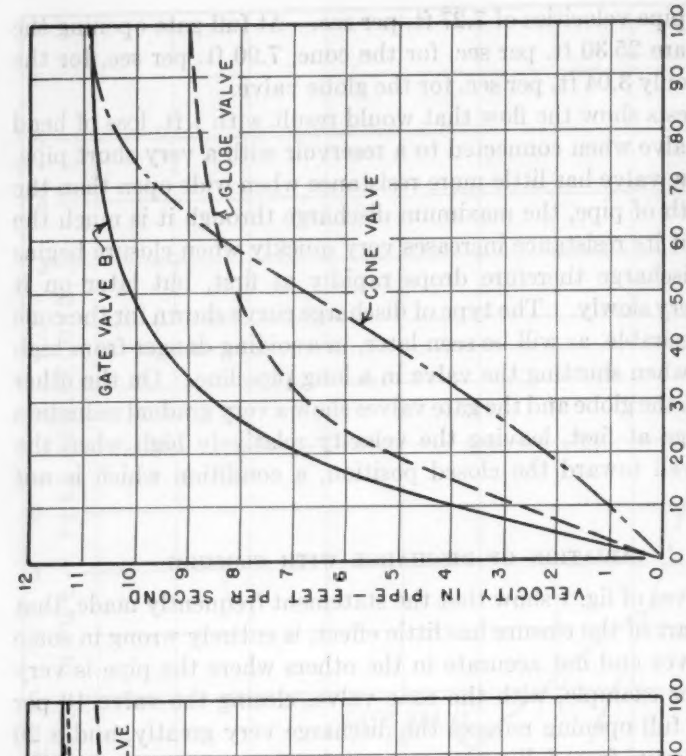


FIGURE 2. FLOW THROUGH NEW 10-INCH PIPE WITH DIFFERENT VALVES AND VALVE OPENINGS; PIPE HAS ROUNDED ENTRY AND THE VALVE IS CLOSE TO THE DISCHARGE END OF THE PIPE (SEE FIGURE 1)

the action of each valve when used to control the flow in a long pipe line. In order to examine this feature, two very simple systems have been assumed, the first of which consists of 2,000 ft. of new horizontal 10-inch pipe, under a 100-foot head, connected by bell-mouthed entrance to the vertical side of a tank, and having a valve at the discharge end of this pipe to control the flow which is directly into the atmosphere. The second system is exactly like the first, except that the pipe is only 500 ft. long and the head is 25 ft. in order to produce nearly the same maximum velocity as in the former case. If these arrangements appear unduly simple, they will, at least, explain what happens in the pipe when the valve is being closed, and serve as a guide as to what may be expected in more complicated piping.

RESULTS FROM LARGE GATE VALVES

The author did not have available results similar to fig. 1 for large valves, except in the case of gate valves. In this latter valve, numerous tests have been made and the calculations following are based largely on results given in "Hydraulics for Engineers" by the author, based on tests by Professor Corp; these results show a lower resistance for a given setting than that corresponding to the 3-inch valve. The valve having coefficients corresponding to those of the 3-inch one has been designated as valve "A" and that having coefficients corresponding to results on larger valves is marked valve "B." In the case of the cone and globe valves the coefficients obtained for the 3-inch valve are assumed to hold for the 10-inch also, and some tests on a 6-inch cone valve show this to be nearly true for it.

On fig. 2 (a) are shown curves of discharge (stated in terms of pipe velocity) through the first system described above, i.e. with 2,000 ft. of 10-inch new pipe under a 100-foot head. The three valves were assumed attached one at a time and the pipe flow is given on a base of per cent of closure of the valve. On fig. 2 (a) two gate valve curves are shown, one for gate valve A, the other for gate valve B. On fig. 2 (b) similar curves are shown for the second system, i.e. 500 ft. of 10-inch pipe with a 25-foot head.

The behavior of the valves is most striking. Referring first to fig. 2 (a), gate valve B may be half closed and only causes a reduction of pipe velocity from 11.3 ft. per sec. to 11.1 ft. per sec., an almost negligible result, and even when this valve is 80 per cent closed the pipe velocity is still 84 per cent of the flow for full open valve. In

other words, nearly all of the velocity in the pipe is extinguished during the last 20 per cent of gate movement. This alone shows that a uniform rate of closure of this valve is undesirable, and if done quickly may cause high pressures in the pipe; but, on the other hand, it is hardly possible to make the first 80 per cent of movement fast enough to cause undue pressures.

The results are similar for the globe valve but are not quite so marked. Thus, if the globe valve is substituted for the gate valve, the velocity reached for full opening is only 10.7 ft. per sec., and at 50 per cent opening it is 10.1 ft. per sec., a very slight change; at 20 per cent opening it has fallen to about 77 per cent of the full open valve, a result slightly more favorable than with the gate valve. In both the gate and globe valve it is fairly accurate to state that the greater part of the valve movement has little effect on the discharge. The statements would be less accurate for fig. 2 (b), for in the shorter pipes the valve effects are relatively more important, and in this connection the two parts of fig. 2 are well worthy of study and comparison.

Referring next to the cone valve (or any other type of valve where the resistance increases rapidly at first when the valve begins to close) the effect is notably different; the velocity in the pipe begins to change immediately the valve is moved. It gives the same maximum velocity as the gate valve B in this case, and at 50 per cent closed the velocity has dropped to 83 per cent of the initial value, and from 40 per cent opening down, the slope of the velocity curve is only about one-third that for valve B. Such a valve will therefore produce a much smaller pressure rise for uniform rate of closure than the gate or globe valves would. The argument applies with equal force to fig. 2 (b), though all the velocity curves are flatter than in the former case, due to the relatively greater effect of the valve in the shorter pipe.

In comparing the different curves in fig. 2, it must be remembered that in the left-hand figure the sole difference in the cases is in the valves used, the system and head being otherwise the same for all. The same is true for the right-hand figure.

EFFECT ON WATER HAMMER PRESSURES

A study of the effect of the rate of closure on the water hammer pressures produced was then made by the method described in

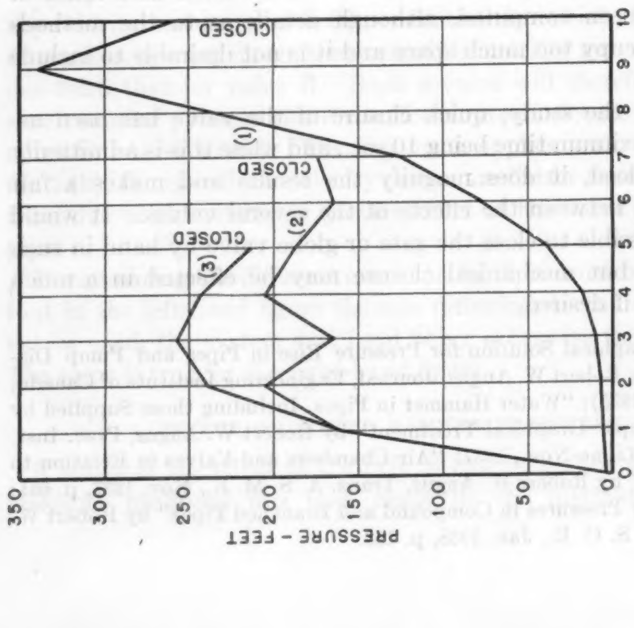
previous papers by the author.* As would be expected, none of the valves cause any trouble up to the time they are reduced from full to half open, no matter how quickly the movement is made, although the cone valve shows a slightly greater rise than the others. But if the rate of closure is kept uniform from full open to the closed position, there is a marked pressure rise which increases rapidly as the complete closure is very closely approached, the cone valve showing a smaller pressure rise than the others for this method of closure.

STUDY WITH 10-IN. PIPE LINE

In order to get specific values for the pressure rise due to gate closure, a study was made of the pipe line considered on fig. 2 (a), with one gate valve "B" and then with the cone valve, and finally with the globe valve. When pressure rise occurs due to decrease of discharge, it shows itself from one end of the line to the other, the pressure "wave" traveling up and down the pipe, in this case at the rate of 4,000 ft. per sec., so that the period of the pipe, i.e. the time required for the wave to travel from the gate to the reservoir and back again to the gate, a total distance of 4,000 ft., would be 1 second. It is readily shown that the maximum pressure rise will occur at the valve which causes the velocity change, so that it has not been thought desirable in this case to examine any other point. The pressures at the gate due to different methods and times of closure have been computed, although details as to the methods used would occupy too much space and it is not desirable to include them here.

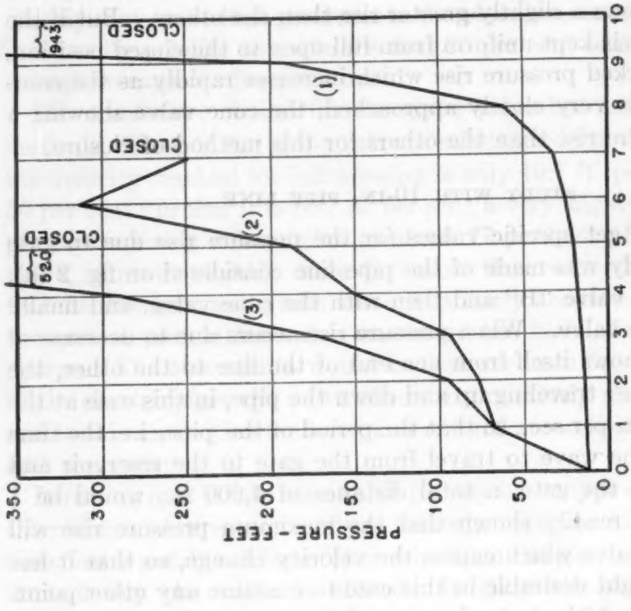
Throughout the study, quick closure of the valve has been assumed, the maximum time being 10 sec., and while this is admittedly very short indeed, it does magnify the results and makes a fair comparison as between the effects of the several valves. It would be quite impossible to close the gate or globe valve by hand in such a short time, but mechanical closure may be effected in a much shorter period if desired.

* "Simple Graphical Solution for Pressure Rise in Pipes and Pump Discharge Lines" by Robert W. Angus, Journal, Engineering Institute of Canada, **18**: 72 and 264 (1935); "Water Hammer in Pipes, Including those Supplied by Centrifugal Pumps—Graphical Treatment" by Robert W. Angus, Proc. Inst. M. E., **136**: 245 (June-Nov., 1937) "Air Chambers and Valves in Relation to Water Hammer" by Robert W. Angus, Trans. A. S. M. E., Nov. 1937, p. 661; "Water Hammer Pressures in Compound and Branched Pipes" by Robert W. Angus, Proc. A. S. C. E., Jan. 1938, p. 133.



(1) CLOSURE AT UNIFORM RATE IN 10 SECONDS
(2) 70% CLOSURE IN 1 SEC. THEN UNIFORMLY IN 6 SEC.
(3) 70% " 1 " " " 4 "

FIGURE 3. CONE VALVE, PRESSURE RISE AT VALVE DURING CLOSURE FROM FULL OPENING, 2000 FEET OF 10-INCH PIPE, 100 FOOT HEAD.



(1) CLOSURE AT UNIFORM RATE IN 10 SECONDS
(2) 80% CLOSURE IN 1 SEC. THEN UNIFORMLY IN 6 SEC.
(3) 80% " 4 " " " "

FIGURE 4. GLOBE VALVE, PRESSURE RISE AT VALVE DURING CLOSURE FROM FULL OPENING, 2000 FEET OF 10-INCH PIPE, 100-FOOT HEAD.

CONE VALVE ATTACHED TO 2000 FT. OF PIPE UNDER 100-FOOT HEAD

The results of the study of this case are shown on fig. 3. No attempt has been made to round off the corners of the diagrams. Curve (1) shows the effect of closing the valve at uniform rate in 10 sec. and starts with a very low pressure at the gate at full flow on account of the large friction loss. The pressure rise at the gate is gradual and reaches the reservoir pressure in 6.5 sec., but at this time it is rapidly rising and reaches 340 ft. in 9 sec., after which it falls to 261 ft. at gate closure in 10 sec. The maximum pressure is thus 3.4 times the static head.

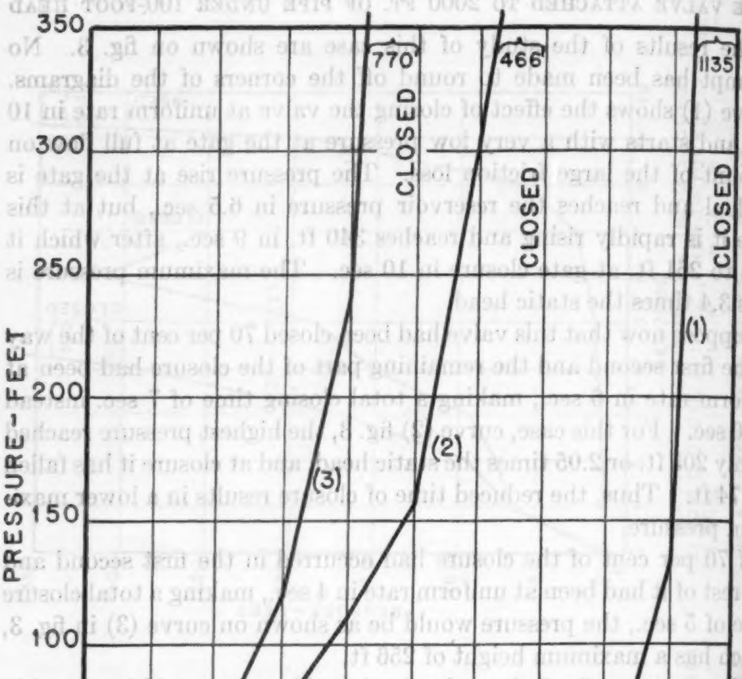
Suppose now that this valve had been closed 70 per cent of the way in the first second and the remaining part of the closure had been at uniform rate in 6 sec., making a total closing time of 7 sec. instead of 10 sec. For this case, curve (2) fig. 3, the highest pressure reached is only 205 ft. or 2.05 times the static head, and at closure it has fallen to 174 ft. Thus, the reduced time of closure results in a lower maximum pressure.

If 70 per cent of the closure had occurred in the first second and the rest of it had been at uniform rate in 4 sec., making a total closure time of 5 sec., the pressure would be as shown on curve (3) in fig. 3, which has a maximum height of 256 ft.

The curves on fig. 3 show clearly that this valve could be closed in 5 sec. to 7 sec. without danger, a statement scarcely credible, and, further, that the longer time of closure is actually more dangerous than the shorter time, if the latter is under proper control.

The curves on fig. 4 give a graphical representation of a similar study to the above, but with a globe valve instead of the cone valve. Closure of this valve at a uniform rate in 10 sec., curve (1), produces a total pressure of 943 ft., which means that this method of closure could not be permitted with safety. Next, closure times of 7 sec. and 5 sec. were tried, but both with the globe and the gate valve initial closures of 80 per cent in 1 sec. were used instead of 70 per cent as with the cone valve. Without reproducing the calculation to prove the statement, it is a fact that 80 per cent closure in this case is much more favorable to the valve than 70 per cent would have been, and for this reason the 80 per cent closure in the first second was adopted.

For a total closure time of 7 sec., curve (2) fig. 4, the maximum pressure reaches the value of 313 ft., while for 5 sec. closure, curve (3) of the same figure, shows that the pressure reaches the dangerous value of 520 ft. Evidently this valve causes much higher pressures than the cone valve if operated under similar conditions.



(1) CLOSURE UNIFORMLY IN 10 SECONDS
(2) 80% CLOSURE IN 1 SEC., THEN UNIFORMLY IN 6 SEC.
(3) 80% " " 1 " " " 4 "

FIGURE 5. GATE VALVE B. PRESSURE RISE AT VALVE DURING CLOSURE FROM FULL OPENING. 2000 FEET OF 10-INCH PIPE., 100 FEET HEAD.

GATE VALVE ATTACHED TO 2000 FT. OF PIPE UNDER 100-FOOT HEAD

The results of the pressure rise computations for this case are shown on fig. 5, and conditions similar to the former examples were used. This valve shows a markedly different effect from the others, for when closed at uniform rate in 10 sec., the pressure reaches the highest value of any of the three, viz. 1,135 ft., which would doubtless rupture a pipe properly designed for a working pressure of 100 ft. If this valve is closed 80 per cent of the way in the first second and the balance of the closure is effected at uniform rate in 6 sec., giving a closing time of 7 sec., then curve (2) fig. 5 shows a maximum pressure of 466 ft. and finally, if the total closing time is reduced to 5 sec., with 80 per cent of it in the first second, the pressure at closure would be 770 ft.

Under any of the methods of operation described, the gate valve would produce dangerous pressures, and for it the time of operation would have to be much increased. Indeed, a study of fig. 2 (a) shows that as far as safety of operation is concerned, the cone valve is the best because the *rate of change of velocity* with valve closure is much less in it than in the others; the globe valve would be next in order and the gate valve last, thus confirming what the water hammer calculations prove.

It would have been instructive to have carried through the complete analysis for the cases shown in fig. 2 (b) where only 500 ft. of pipe are used and where the pressure has been reduced to give roughly the same velocity at full open gate as in the other cases. However, only one case has been worked out, which will be sufficient to show the need of care in this case also. Gate valve B has been selected and the case is where 80 per cent of the closure has been made in the first second and the balance of it at uniform rate in 4 sec. The example may therefore be compared with fig. 5 curve (3). But one would expect much different results, for in the case of fig. 5 the pipe velocity change for an 80 per cent closure would, according to fig. 2 (a), be about 1.9 ft. per sec., whereas in fig. 2 (b) with 500 ft. of pipe the change would be slightly under 4 ft. per sec., and this results in a much greater relative rise for the first movement; this is further aggravated by the fact that the pressure rise always shows a marked relative increase when the head is decreased under otherwise similar conditions.

The results for the 500 ft. of pipe are shown in fig. 6, the maximum pressure reaching 130 ft. or 5.2 times the static head, but it then

rapidly drops to more reasonable values. For this case a different method of operation is desirable than for the longer pipe.

These simple examples show:

1. That very rapid closures may be effected with perfect safety, provided the characteristics of the valve are suitable and that it is operated with sufficient care, which in most cases need not involve any special mechanism.

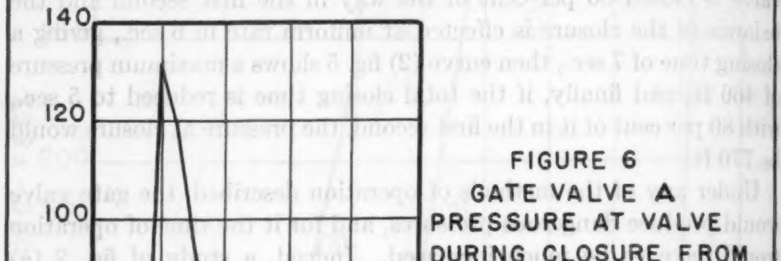


FIGURE 6
GATE VALVE A
PRESSURE AT VALVE DURING CLOSURE FROM FULL OPENING. 500 FT. OF 10-INCH PIPE, 25-FT. HEAD

2. That the valve which is safest from water hammer point of view has very low resistance when full open, but has a rapidly increasing resistance as closure begins. Such a valve immediately has an effect on the discharge when closure begins because the losses in it are an appreciable proportion of the total losses in the system. The resistance of gate valves down to even 40 per cent closed is not large, amounting to only about 10 per cent of the total loss on the long pipe

and 33 per cent on the short pipe for that setting, while the corresponding figures for the cone valve are 48 per cent and 80 per cent.

These cases are simply illustrations to show the need of a careful examination of each case where quick closure may be desirable, as in case of burst pipes. Further, there are other valves with similar characteristics to the cone valve examined, but such were not available at the time of the tests.

CHECK VALVES

Space prevents a discussion of the properties of check valves, particularly on the discharge side of pumps. These valves may perform their final closing movement in a fraction of a second, producing dangerous pressures in addition to the slamming action on the valve seat. The cases illustrated show that the most rapid pressure rise frequently occurs just when the valve is finally closing, showing the dangerous effects of the uncontrolled valve. Dash pots and control weights are used with good effect, if properly applied, but a return of the pressure wave may produce a negative pressure at the valve, causing it to open after its first closure, in general a rather dangerous thing that should be prevented. Sometimes air chambers are used on the lines for protection against the water-hammer pressure produced by the check valve, but these chambers must be large to be effective and do not provide a desirable solution to the problem, and it is better to so design the valves as to prevent the pressures reaching dangerous limits.

THE WATER SUPPLY OF GREATER WINNIPEG

By W. M. Scott

Before the Greater Winnipeg Water District came into being, the city of Winnipeg supplied water to five of the municipalities now within the District, and the neighboring city of St. Boniface supplied one. In this way eight of the nine municipal units which now comprise the District had already some experience in coöperation and this probably facilitated the solution of problems of organization, finance, law and politics, which frequently are more difficult than those of engineering in creating a utility of this kind.

The city took the iniative in establishing the metropolitan District of which it represents about 46 per cent in area, 84 per cent in water consumption and 86.5 per cent financially. The following observations regarding Winnipeg's location, growth in population, and first water supplies are preliminary to the description of the Greater Winnipeg Water District. The city is located in the basin of an ancient glacial lake, known as Lake Agassiz. This lake extended as far south as the head waters of the Red River and as far north as the Nelson River, included Rainy Lake and Lake of the Woods on the east side, Lake Manitoba and Lake Winnipeg on the north, and extended to the Pembina Mountains on the southwest. Beneath the city there is a deposit of sedimentary clay varying in thickness from 40 to 60 feet and interspersed with strata of sand and gravel, some of which are water bearing and contain, in all probability, a rather high percentage of limestone. Below the gravel lies limestone rock.

The city is situated about 60 miles north of the international boundary where the Red River (which rises in the state of Minnesota and flows northward) joins the Assiniboine River coming from the west. Before the city's incorporation in 1873, it was a small trading post known as Fort Garry and was the scene of stirring events in that chapter of Canadian history known as the Red River Rebellion.

A paper presented at the Canadian Section meeting, Windsor, Ont., March 25, 1938, by W. M. Scott, Chairman of Commissioners, Greater Winnipeg Water District, Winnipeg, Man.

The population of Winnipeg increased rapidly from 1874, when it was 1,869, until 1915 when it was 212,889. War was probably only a minor reason for the decline in 1915; opening of the Panama Canal in 1915 is more likely the cause. Prior to the opening of the Canal, Winnipeg was the chief distributing point for territory reaching westward almost to the Rocky Mountains. Water freight effected such a change in this tributary territory that it now does not reach even to the western boundary of Manitoba. It was 1931 before the population was again equal to that of 1915, and by 1937 it had increased to only 224,533 (see fig. 1).

The 1907 report of the Board of Engineers regarding prospects for a new water supply for the city predicted that the population would be somewhat greater than 500,000 in 1937. Actually, for the city and adjoining municipalities included, the population is somewhat less than 300,000. The report was made in good faith and according to an approved method, but influences which were not foreseen, changed radically the actual growth from the expected.

The source of the first water supply was the Assiniboine River. In 1880 a twenty-year franchise (which terminated December 23, 1900) was granted to the Winnipeg Water Works Company, a private enterprise. About four miles of mains were laid and water was turned on in July 1882. The population of the city at that time was about 16,500.

The Assiniboine River supply was never satisfactory. Although the water was filtered through pressure filters, there was always a suspicion of organic pollution. The water for the greater part of the year had a disagreeable taste which could not be removed by filtering. In spring, summer and autumn the stream was so turbid that the filters were almost continually clogged.

The city in 1897 retained the late Dr. Rudolph Hering who, after a study of the situation, recommended that wells be used as a source of supply, with provision for softening part of the supply. Winnipeg built, in 1901, the first municipal water softening plant on this continent, using lime-soda ash. In 1903 the first municipal plant in the U.S.A. was built at Oberlin, Ohio.

In 1899 the city purchased the plant of the Winnipeg Water Works Company, making it a municipally owned and operated utility and, adopting the recommendation of Dr. Hering, changed the source of supply to wells. Between 1900 and 1908 a group of seven wells was dug. These averaged about 18 feet in diameter and varied in

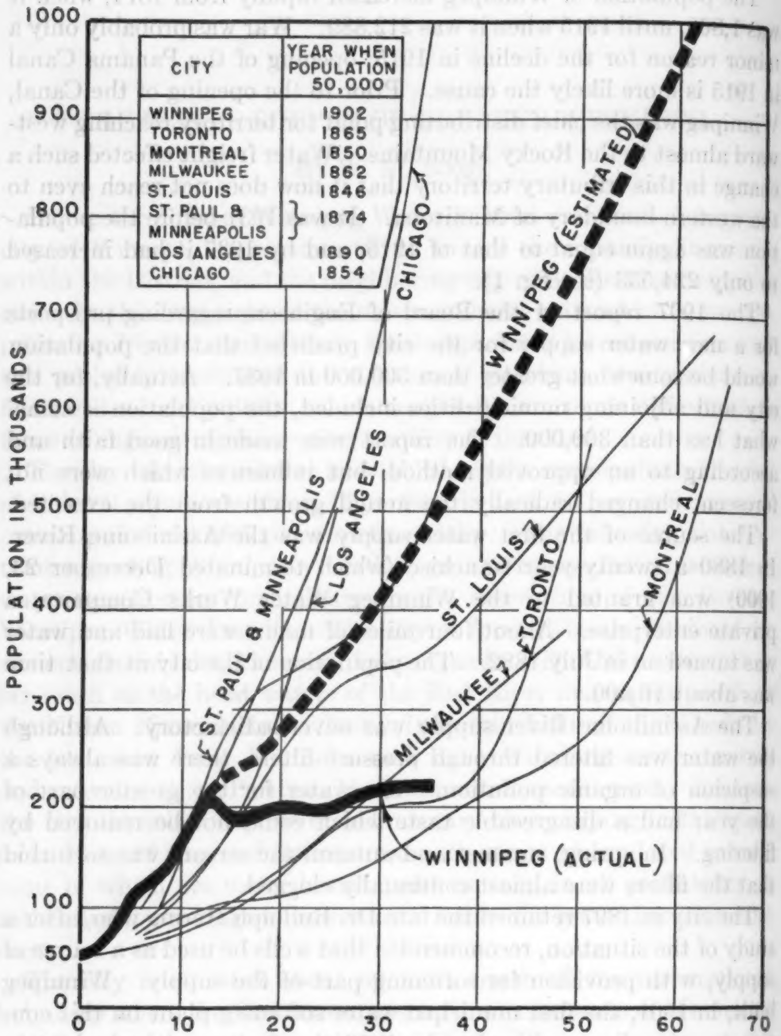


FIGURE 1. CITY OF WINNIPEG

1907 FORECAST OF GROWTH IN POPULATION

COMPARED WITH ACTUAL GROWTH

depth from 46 to 102 feet. Later, before the well system was abandoned, a series of 18-inch wells was drilled, extending northward from the city for several miles.

From a sanitary standpoint the quality of the ground water seemed to be satisfactory. Frequent analyses showed no indication of contamination. Its physical qualities were also satisfactory. It was clear, colorless, odorless and cool. Its hardness and salinity gave it a marked taste quickly noted by those not accustomed to

TABLE 1

Analyses of water from three sources

	ASSINIBOINE RIVER	WELL WATER	SHOAL LAKE
	parts per million	*parts per million*	*parts per million*
Color	16		16
Turbidity	12		Slight
Odor	Slightly vegetable		Very faint vegetable
Taste			Slightly "
Ammonia—free	0.15		.068
Ammonia—albuminoid	0.08		.40
Nitrites	0.19		Trace
Nitrates	0.20		.1
Oxygen consumed	5.10		1.8
Total solids	605	1014	130
Calcium	121	94	26
Magnesium	41	68	7
Sodium	34		2
Chlorine	26	246	1
Sulphate	163	163	2
Carbonates	195		54
Alkalinity	325	360	90
Incrustants	145	158	Nil
pH	7.7	7.8	7.4

its use but not objected to by those familiar with it. It was excessively hard (see table 1), not a good water for domestic use and in many cases impossible to use for industrial purposes. Adequate softening was not feasible. While no contamination had ever been detected, there was danger should increasing demand result in the wells being drawn down by pumping below the level of the rivers.

Also, the ground water was never sufficient to furnish an adequate supply at good pressure for domestic use alone; and the deficiency was still more marked in respect to fire protection. In view of this

situation and the rapidly increasing demand by the existing and prospective industrial plants, it was decided to develop a suitable supply.

The Water Supply Commission of the City of Winnipeg was established by a special Act of the Manitoba Legislature in 1906 for the purpose of taking the necessary steps to develop a supply in every way adequate for the city's needs. The Commission engaged a Board of Consulting Engineers who, on August 29, 1907, submitted a report summarized as follows:

> "Finally considering the quality of the water from the various sources, the cost of the works, the time required for construction and the relative advantages and disadvantages of each project, we recommend that the city go to the Winnipeg River for its future water supply."

At this time the population of the City was about 111,000. The Commission, on October 30, 1907, recommended to the City Council that the report of the consulting engineers be acted upon. The city at this time, however, had already undertaken its hydro-electric development at Pointe du Bois on the Winnipeg River and was committed to the expenditure of about six million dollars for that purpose. Chiefly for this reason the water supply project was postponed for a time.

The hydro-electric development was completed in the fall of 1911 and in 1912 the consideration of a permanent water supply was renewed. During 1912 the Public Utilities Commission, at the request of the city, made further studies regarding a new source of supply. These pointed to Shoal Lake as the most desirable source. When the matter came before the Public Utilities Commissioner in 1912, the report approving the scheme also contained the suggestion that adjacent municipalities might join in procuring this water for the benefit they would naturally get from the supply.

Winnipeg decided in favor of the scheme. The city of St. Boniface coöperated with Winnipeg and a bill was drafted creating what was at first designated as "The Winnipeg and St. Boniface Water District." While this matter was being discussed, other adjoining municipalities held a joint meeting on January 17, 1913, and they requested that they be admitted as participants in the plan. At that meeting (at which Winnipeg was represented) it was decided that those of the municipalities which intended to coöperate with Winnipeg and St. Boniface should pass formal resolutions approving the

scheme and expressing their intention to become parties thereto. Pending receipt of these resolutions, a new draft of the bill was prepared providing for inclusion of the name and area of each municipality which was to become a partner. In due course all the municipalities interested passed the resolution required and were admitted into partnership, and in accordance with the terms thereof, the bill was drafted in final form.

The municipalities thereafter gave the necessary information describing the area of the municipalities comprising the District, the population of each municipality, and the assessments. This information seems to have been the basis for arriving at the proportion of the cost which was to be borne by each municipality and was also the basis of the whole legislative contract which was made in the form of the Greater Winnipeg Water District Act.

On April 7, 1913, the City Council appointed a Board of Consulting Engineers who were instructed to submit a report upon the best means of supplying the Greater Winnipeg Water District with water from Shoal Lake, together with an estimate of cost and general plans of the work. The report of this Board was submitted on August 20, 1913, and contained the following major recommendations:

That Shoal Lake water is of excellent quality for domestic and for manufacturing purposes; that Shoal Lake can be depended upon to furnish all the water required for the Greater Winnipeg Water District until the population shall have reached about 850,000 and with the help of the Lake of the Woods can furnish a practically inexhaustible supply; that this water be brought from Shoal Lake through a covered concrete aqueduct 84.5 miles in length (under open flow conditions to within about 8 miles from St. Boniface, thence through a 5.5-foot reinforced concrete circular pipe under pressure to the eastern bank of the Red River, thence through a 5-foot cast-iron pipe in tunnel under the river) and thence through a 4-foot reinforced concrete pipe to the city reservoirs at McPhillips Street; the total length of the aqueduct to be 96.5 miles.

AQUEDUCT DESCRIBED IN 1920 JOURNALS

The structures as finally built conformed very closely to the Engineering Board's recommendation. The résumé below gives pertinent data on the Shoal Lake supply and general facts relating to the aqueduct. A comprehensive description of the aqueduct, giving general features, diagrams, and a map, is to be found in an article by

the late J. H. Fuertes in the September 1920 issue of the JOURNAL. An article by the late W. G. Chase in the November 1920 JOURNAL deals chiefly with the details of construction. There are certain features of the general plan, however, which may be reviewed here.

Lake of the Woods is an international as well as an interprovincial body of water, therefore it was necessary to apply to the Inter-

---

RÉSUMÉ

Data on the Shoal Lake Water Supply

Preliminary estimate of cost of the undertaking, exclusive of land and interest during construction	$13,045,000
Source of supply: Indian Bay, Shoal Lake and Lake of the Woods.	
Area of Shoal Lake	107 sq. mi.
Area of Lake of the Woods, including Shoal Lake	1,500 sq. mi.
Drainage basin of Shoal Lake	360 sq. mi.
Drainage basin of Lake of the Woods	27,700 sq. mi.
Difference in elevation between Intake and Winnipeg Reservoir	290 ft.
Method of delivery—gravity	
Area of Greater Winnipeg Water District	53.8 sq. mi.
Population of Greater Winnipeg Water District in 1937	285,000
Length of cut and cover concrete aqueduct, 85 m.g.d. capacity	77.5 mi.
Length of river syphons and pressure section, 85 m.g.d. capacity	7.1 mi.
Length of reinforced concrete pressure pipe, 50 m.g.d. capacity	9.4 mi.
Length of Red River tunnel with 5-foot cast-iron lining	0.2 mi.
Length of 4-foot concrete pipe in streets of Winnipeg	2.3 mi.
Date work was begun on undertaking	October 1, 1913
Date set for completion	October 31, 1918
Water turned into McPhillips Street reservoir	March 29, 1919
Time (estimated on a 30 m.g.d. flow) for water to flow from intake to Winnipeg reservoir	51 hr.
Equalized Assessment for 1937	$102,155,000.00
Equalized Assessment for 1938	$82,444,000.00
Levy for 1927	$820,000.00
Levy for 1938	$795,000.00
Bonded Indebtedness, Dec. 31, 1937	$16,542,112.83
Sinking Fund Assets, Dec. 31, 1937	$4,397,512.81
Average consumption of water during 1937 within Greater Winnipeg Water District in gallons per head per day	64.5

---

national Joint Commission for approval of a diversion of water from Shoal Lake, Ontario, for the District's needs. Hearings were held in 1913, and in January 1914 the right was granted to the District to divert a maximum of one hundred million gallons per day. The levels of the Lake are maintained between elevations 1056 and 1061 (Geodetic Datum) by the Lake of the Woods Control Board.

There is virtually no settlement on the shores of the Lake of the Woods; there are only a few Indians part of the year. There is no filtration or other treatment given to the District's water supply with the exception of a small dose of chlorine at Winnipeg and at St. Boniface. During the winter season this is about 0.4 p.p.m.—sufficient to show a residual of 0.1 p.p.m. During the spring turnover, this is increased to about 0.8 p.p.m. and later again reduced.

To build the aqueduct there was built 110 miles (including sidings) of standard railway. This has been retained and operated continuously since 1920. It's chief function is that of a patrol road, but it gives a passenger, freight and mail service to an increasing number of settlers in the area traversed.

FALCON RIVER DYKE DIVERTS HIGH-COLORED WATER

The intake structure takes water from Indian Bay which is the west arm of Shoal Lake which in turn is the western end of the Lake of the Woods. Indian Bay extends into Manitoba (from Ontario) about two miles. A small stream, the Falcon River, empties a highly colored muskeg water into Indian Bay very near to the intake. Indian Bay when first examined, was decidedly discolored throughout its whole area, the greater part of the discoloration coming from Falcon River. To obtain a low colored water and avoid the expense of extending the intake into the Bay, the District built a dyke across the west end of Indian Bay and also a short canal at its southerly end which diverted the Falcon River water into an adjoining body of water, Snowshoe Bay. This scheme has proved completely successful in obtaining a low colored water. The diversion dyke was completed in 1914 and on June 7, 1915 the color of the Falcon Bay water was 107 while the color on the Indian Bay side of the dyke on the same day was 9 (platinum cobalt scale).

The cost of the dyke and canal was $147,000. The estimated cost of extending the intake into Indian Bay was one million dollars.

There are eight river crossings. These are in general inverted syphons of reinforced concrete built in the trench and designed with a

loss in head conforming to the average loss in head for the same distance in the horseshoe shaped sections which they connect.

The Red River is crossed with cast-iron pipe 5 ft. in diameter set within vertical shafts lined with concrete and within a concrete lined horizontal rock tunnel 10 ft. square. The depth of the tunnel below ground surface is about 80 feet.

SURGE TANK PROTECTS SECTION OF AQUEDUCT

Upon the aqueduct on the east bank of the Red River near the tunnel crossing is a reinforced concrete structure, circular in plan and containing a central well 25 ft. in diameter into which the aqueduct discharges. Its overflow weir is at elevation 785.5, the elevation of the invert at the west end of the open flow section and at a distance of thirteen miles, being 791.7. This well is inside of and concentric with a second concrete well 32.5 ft. inside diameter. Its top carries up to support a reinforced concrete roof, the underside of which is 9.67 ft. higher than the overflow lip. The water overflowing the inner circular well escapes to the river through a 36-inch pipe line.

Surrounding the whole structure and separated from it by an annular space 2.75 ft. wide at the bottom reducing to 9.5 in. at the top, is a brick facing with stone base, belt courses and cornice. The function of the surge tank is to protect the circular reinforced concrete pressure section of the aqueduct extending eastward from St. Boniface from excessive pressures which might result from manipulation of the valve which controls the discharge of water into the city's reservoirs.

The Aqueduct as built has not the same discharging capacity for its whole length. It cannot deliver to the Winnipeg reservoirs at present the 85 m.g.d. for which its main section is designed.

The open flow section which is for the most part of plain concrete and extends westward from the intake for about 80 miles has a discharging capacity of 85 m.g.d. From the west end of this horseshoe shaped section there is a depressed section of reinforced concrete 8 ft. in diameter 4 miles long continuing the discharging capacity of 85 m.g.d. to Deacon, the site chosen for a 250 million gallon reservoir in the future. Deacon is about 8 miles east of St. Boniface and about 9.5 miles from the surge tank on the east bank of the Red River. Between these points there are approximately 49,000 lineal feet of 5.5-foot lock joint pipe. This has a discharging capacity of about 50 m.g.d.

A 5-foot tunnel crosses the Red River and a 4-foot lock joint reinforced concrete pipe 11,659 ft. long extends through the city of Winnipeg to its reservoirs in the western end. This latter pipe has a capacity of 28.5 m.g.d. under existing conditions.

When the demand from the city reservoirs exceeds this rate, which will be approximately when the average consumption reaches 20 m.g.d., a pumping station will have to be established at the surge tank capable of delivering up to 50 m.g.d. to the city reservoirs. When the demand approaches 50 m.g.d., a second 5.5-foot pipe from Deacon to St. Boniface will be required. Possibly before that time, the proposed reservoir at Deacon will be built.

The water elevation at the intake averages about 290 ft. above that in the city reservoirs. To have made use of this head or a part of it to deliver water to consumers at their taps would have required an aqueduct of very much more costly construction than the one built and would also have been more difficult to operate. As Winnipeg enjoys cheap electric power, the water is pumped from the receiving reservoir to consumers. The economics of this method are believed to be sound.

DAMAGE FROM ALKALI ELIMINATED BY SUBDRAINAGE

Parts of the aqueduct, in general confined to that portion from 8 to 12 miles east of St. Boniface, after being in the ground about two years were found to have suffered from contact with the ground waters bearing sulphates of calcium and magnesium. The action is a softening one and gradual disintegration results. The damage occurred in spots, not continuously. It was assumed that a dense hard concrete carefully placed and with a smooth surface would resist the disintegrating action of the soil if the surface waters were properly drained off. This, however, was not sufficient to ensure complete freedom from damage and a certain amount of subdrainage was necessary.

In studying the problem of protection of concrete from these sulphates, it was found that a good concrete is not affected by a concentration of less than 500 p.p.m. Above that point danger increases and is certain, no matter how good and dense the concrete is, when concentrations greatly exceed 1,000 p.p.m. Subdrainage, properly done and maintained, gives effective protection and has done so in the case of this aqueduct.

FEATURES OF GREATER WINNIPEG WATER DISTRICT ACT

According to the Act as drawn, the principle of taxation upon land values only, exclusive of improvements, is observed. The sum of money necessary to pay interest and sinking funds is levied by an annual rate upon all of the lands within the District. This unusual method of levying on land only for carrying charges probably developed from extremely rapid increase in population and in land values during thirty years prior to 1913. It has since been realized that this burden is too heavy upon land. The Act provided that in addition to the levy, a direct charge should be made for water—the same rate to each municipality—such charge as nearly as possible to be arrived at on the basis of cost of maintenance, operation and management of the undertaking. This rate, as at first fixed, was one and one quarter cents per one thousand gallons. It was soon realized that a certain amount of relief should be given on the land tax and this compensated for by a higher direct charge for water. Consequently, an amendment to the Act was secured by which the direct charge was raised to five cents per one thousand gallons.

This increase has been put into effect very gradually as shown in fig. 2. The full five cent rate became effective only on January 1, 1938. The result has been a substantial decrease in land levy. Reduction in the land levy was begun in 1923 when $42,262 was transferred from revenue from sale of water. As the charge per 1,000 gallons increased, the amount transferred to reduce the land levy increased, until an estimated $248,913 will be transferred in 1938.

Each municipality has its own distribution system and buys its water from the District in bulk which means without pressure. The District, as a wholesaler, delivers water to a point on the municipal boundary nearest to the aqueduct at a price which, as already stated, has increased from $1\frac{1}{4}$ to 5 cents per 1,000 gallons. This increase in price has resulted in reduction of the levy from its high point in 1922 of $1,242,190.78 to $795,000.00 for 1938. This method has been successful in transferring part of the burden from the land to the water user.

Under its Act, the District may and does use the distribution mains of the cities of Winnipeg and St. Boniface for conveying water to adjoining municipal boundaries since only two of the latter (St. Boniface and Transcona) in addition to Winnipeg, have a direct aqueduct connection. The District pays five cents per 1,000 gallons for the use of mains for this purpose.

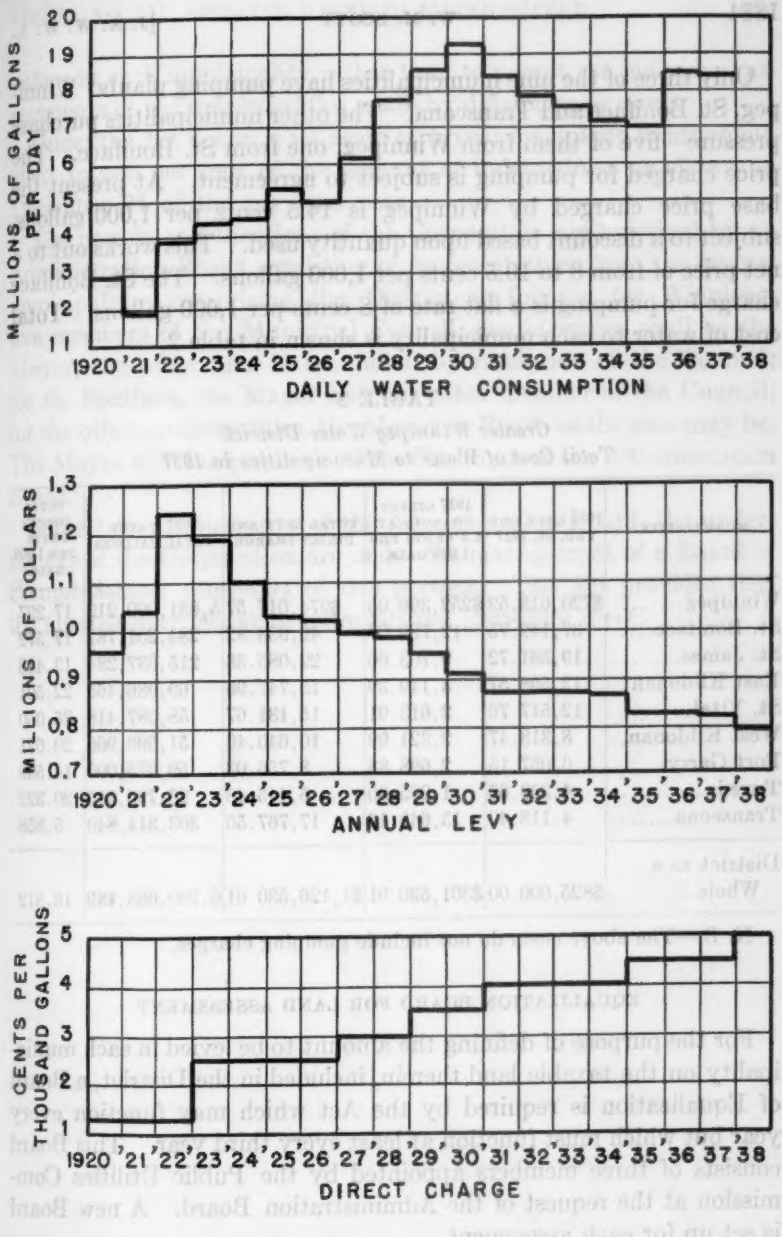


FIGURE 2. OPERATION RECORDS 1920–1937
GREATER WINNIPEG WATER DISTRICT.

Only three of the nine municipalities have pumping plants: Winnipeg, St. Boniface and Transcona. The other municipalities purchase pressure—five of them from Winnipeg, one from St. Boniface. The price charged for pumping is subject to agreement. At present the base price charged by Winnipeg is 14.5 cents per 1,000 gallons, subject to a discount based upon quantity used. This works out to a net price of from 8 to 10.5 cents per 1,000 gallons. The St. Boniface charge for pumping is a flat rate of 8 cents per 1,000 gallons. Total cost of water to each municipality is shown in table 2.

TABLE 2

Greater Winnipeg Water District

Total Cost of Water to Municipalities in 1937

MUNICIPALITY	1936 LEVY DUE FEB. 28, 1937	1937 DIRECT CHARGES AT 4.5 CENTS PER 1,000 GALS.	TOTAL LEVY AND DIRECT CHARGE	1937 WATER TOTAL GALLONS	TOTAL COST IN CENTS PER 1,000 GALS.
Winnipeg........	$720,618.52	$252,399.05	$974,017.57	5,631,090,219	17.297
St. Boniface.....	37,149.75	12,789.07	49,938.82	284,201,782	17.572
St. James.......	19,381.72	9,703.66	29,085.38	215,637,284	13.488
East Kildonan..	12,598.57	3,149.39	15,747.96	69,986,465	22.501
St. Vital........	12,517.76	2,613.91	15,131.67	58,087,418	26.050
West Kildonan..	8,318.47	2,321.99	10,640.46	51,599,966	20.621
Fort Garry......	6,057.15	2,698.88	8,756.03	59,975,000	14.599
Tuxedo.........	4,239.66	1,205.80	5,445.46	26,795,515	20.322
Transcona.......	4,118.40	13,649.16	17,767.56	303,314,840	5.858
District as a Whole.........	$825,000.00	$301,530.91	$1,126,530.91	6,700,688,489	16.812

N. B.—The above costs do not include pumping charges.

EQUALIZATION BOARD FOR LAND ASSESSMENT

For the purpose of defining the amount to be levied in each municipality on the taxable land therein, included in the District, a Board of Equalization is required by the Act which may function every year but which must function at least every third year. This Board consists of three members appointed by the Public Utilities Commission at the request of the Administration Board. A new Board is set up for each assessment.

As its name indicates, the purpose of the Equalization Board is to place on an equal or uniform basis as nearly as possible, the land

values in each municipality rather than to accept the individual assessments of the municipalities made by different assessors.

Under the Act a sum is set aside each year for sinking funds equivalent to one per cent of the amount of indebtedness as shown on December 31 of the previous year.

The powers and functions of the Corporation are exercised by an Administration Board consisting of representatives from the several municipalities comprising the District as follows: for Winnipeg, five members of the Municipal Council, one of whom shall be the Mayor, the other four to be chosen by resolution of the Council; for St. Boniface, the Mayor and one other member of the Council; for the other municipalities, the Mayor or Reeve as the case may be. The Mayor of Winnipeg is ex officio Chairman of the Administration Board.

Subject to the authority of the Administration Board, the undertakings of the Corporation are under the management of a Board of Commissioners, consisting of two persons. The Act provides that the number may be one or more but not exceeding three.

NEW PUBLICATIONS

Elements of Water Supply Engineering

By Earle L. Waterman*

Every teacher of water supply engineering is always in search of that best single text which may be used simultaneously with lectures and laboratory work. The search then further develops into a contest between comprehensiveness of treatment and size of volume. It is the reviewer's judgment that compromise between these upper and lower millstones is not feasible in the water supply engineering field. The field of water supply engineering now encompasses so many important subjects that the effort to compress them all into a single text which can be carried around by the student is well-nigh hopeless.

The author of the textbook under review, in making this effort at compromise, has done exceedingly well. He safeguards himself in his preface to the first edition by stating that the volume is "in no sense a treatise or a handbook, but is frankly a textbook for study by those who are beginning their study of water supply engineering." He assumes that the reader will have some advance knowledge of applied mechanics, materials of construction and hydraulics. The subject matter which he mentions covers the field of water supply engineering with reasonable thoroughness and good arrangement. The chapter headings cover requirements of municipal water supplies, quantity of water, quality of water, examinations, sources, precipitation, groundwaters and their collection, stream flow, impounding reservoirs (including dams), river and lake intakes, transportation of water, pumps and pumping plants, treatment, chlorination, distribution systems, distribution storage, structural features of the distribution system, operation and maintenance and water works finance. When it is recalled that this vast field is covered in 315 pages of text, with the remainder of the volume devoted to problems and index, it may be readily understood why some criticism may

* Second Edition. Published by John Wiley & Sons, Inc., 1938, 329 pp. Price $3.50.

properly be directed at the balance of treatment and at perhaps too simple development of other important issues. The text is well prepared, simply stated and, within the limits of compromise already referred to, represents a useful simple document.

The reviewer would like, however, to see the author in subsequent editions expand a number of the portions of the text and perhaps omit certain chapters entirely, since their adequate treatment deserves several volumes in themselves. Examples of some of these difficulties are noted briefly below, not in the spirit of criticism, but in the direction of suggestion for future amplification of the volume.

One of the few references in the entire text to hydrogen-ion concentration occurs on page 60 to the extent of seven lines. The reviewer believes that this particular unit of measurement deserves far more space in a water supply text than these seven lines provide, particularly in view of the fact that virtually no comment on this control item occurs in the discussion of coagulation and other forms of water treatment.

It is unfortunate that in an edition appearing in 1938 of a text dealing with water supply, virtually no reference occurs on the very unusual precipitation and stream flow situations which have occurred since 1929 both in low flow and flood flow. The treatment of precipitation and flow in pages 74 to 80 would be substantially strengthened if recognition were given to these critical periods from 1930 to 1938. Similar comment may properly be made on the discussions dealing with maximum rates of rainfall from pages 82 to 84 inclusive.

From the standpoint of balance, is it wise to devote 9 pages to mathematical treatment of the hydraulics of wells and less than 7 pages to chlorination? This, of course, is a matter of individual judgment, but with the student forced into the position of selection, he must be given some guidance as to the weight which individual items in the field of water supply may demand of him at some future date.

In the chapter on impounding reservoirs, greater safety of treatment possibly would suggest that the discussion of dams extending over less than 20 pages might properly be omitted and reference to authoritative texts presented in its place. There is real risk attached in including in a textbook only two paragraphs on spillway capacity (page 151) when problems in connection with this particular question deserve a complete text.

In similar fashion, the chapter on sedimentation aided by coagula-

tion could be expanded with profit, particularly in the discussion of coagulants. Expansion of the discussion in the light of physical chemical development would certainly be helpful.

The reviewer has some curiosity as to how the author determined on the validity of the statement on page 228 that the sand in a rapid sand filter plant "should have an effective size between 0.35 and 0.45 mm." General agreement on this range of size is certainly lacking.

In a subsequent edition the reviewer would like to see the discussion of water-borne diseases on page 30 extended to diseases other than typhoid fever and particularly to a great many of the more mysterious occurrences since 1929. Similar extension would be desirable in the chapter on operation and maintenance, particularly in the discussion of cross-connections, where experience since 1929 should be included.

In the chapter on water works finance, a welcome addition to any brief text, a preliminary statement of exactly how a water supply can be financed would be helpful. The reviewer has some curiosity as to the support for many of the statements on page 311, where the author discusses the relative merits of private and public ownership of water works systems. He opens in that section a highly controversial topic and perhaps does not give the student the benefit of all points of view.

On page 266 where the author discusses the analysis of flow in a distribution system based largely upon the Hardy Cross method, he gives a number of mathematical equations in which apparently a number of errors have crept by the proof reader. It appears that in a number of instances where the plus sign appears, the minus sign should occur. In one or two other instances in equations 1 and 2 omissions of significant subscripts confuse the reader. If these errors do exist they should certainly be carefully checked and corrected in forthcoming reprints.

It is perhaps captious to point out some of these possibilities of extension when the reviewer has already emphasized the difficult task to which the author has subjected himself. On the other hand, if the task is assumed, even a compact text, necessarily brief in character, might in subsequent revisions show improved balance, gradual strengthening in certain directions and perhaps even omission of material which cannot be successfully handled in one or two pages.

ABEL WOLMAN

ABSTRACTS OF WATER WORKS LITERATURE

Key. 29: 408 (Mar. '37) indicates volume 29, page 408, issue dated March 1937. If the publication is paged by issues, 29: 3: 408 (Mar. '37) indicates volume 29, number 3, page 408. Material inclosed in brackets, [ ], is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene (British); C. A.—Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R.—Water Pollution Research (British).*

HYDROLOGY

Drought of 1936. With Discussion of the Significance of Drought in Relation to Climate. John C. Hoyt. U. S. Geol. Survey Water Supply Paper 820 ('38). During each of 5 yrs., '30–'34, except '32, one or more states with exception of Me., Vt., La., Miss., and Ark. experienced a major drought; '35 was normal water-supply yr. but in '36 droughts again prevalent. Drought conditions said to prevail when in area ordinarily humid natural vegetation becomes desiccated or defoliates unseasonably and crops fail to mature owing to lack of pptn., or when pptn. is insufficient to meet needs of established human activities. In humid and semi-acrid states no serious droughts occur unless annual pptn. is as low as 85% of mean or lower. U. S. is divided into 3 sections with respect to pptn.,—humid East where pptn. usually adequate, arid West where pptn. sufficient only for limited amt. of natural vegetation without irrigation, and intermediate semi-arid states where pptn. may make section either humid or arid. 5 of 11 major drought yrs. since 1881 occurred in period '30–'36 in which period min. pptn. records were equalled or broken in 21 states, '36 drought general over humid and semi-arid states, in humid section principal drought area in the Ohio and Miss. river basin and in Mich. In general '36 conditions (in humid area) not as severe as '30 and not especially unusual. Especially severe however in semi-arid states except Texas, with general crop failures and insufficient water supplies. '36 drought caused by deficient and undistributed pptn. accompanied by high temps. and warm winds. Droughts best studied from standpoint of 3 important water supply seasons, —storage period Dec.-Apr. when losses are least; growing period May-Aug. with max. depletion of soil moisture and min. ground water recharge; and replenishment period Sept.-Nov. when accretion to soil moisture and min. ground-water commonly occur. In humid states, '36 pptn. did not exceed 85% of normal in 54% of area of these states and 12 states were affected; av. pptn. was 76% and ranged from 65% to 84%. This deficiency led to reduced crop yields of 60% to 79% of normal. In semi-arid states except Texas, pptn. ranged from 50% to 70%, with previous lows in N. Dak., S. Dak., and Kansas exceeded, leading to crop yields of only 37% to 62% of normal. Pptn. approached or exceeded normal in all arid states except Montana. '36 winter was cold; in summer max. temps. of over 100° were recorded in all states except

3 New Eng. states, and in most cases max. of 110° or more were reported. In semi-arid states, max. in all except Neb. (118°) equalled or exceeded 120°. Severe wind storms in Ga. and La., Ga. having at least 7 major tornadoes in April with 230 deaths and 1500 injured. There were no marked deficiencies in water supplies in '36, either surface or ground, except in semi-arid states. Few records of min. stream flow were broken, no epidemics or unusual health condition attributable to drought of this yr. Using 15% deficiency as criterion there have been 39 droughts since 1880 in humid states and 30 in semi-arid states, in humid section interval between noticeable droughts is 3 to 5 yrs. Judged from both deficiency in pptn. and area affected worst drought yrs. in humid states were in order named, '30, '36 and '94; in semi-arid states, '36, '34 and '10. A number of tables containing data of rainfall, temp. etc. and discussions of affects of drought on sections, agriculture etc. are included in bulletin.—*Martin Flentje.*

Weather Bureau's Contributions to Hydrology. WILLIS RAY GREGG. Civ. Eng. **8**: 323. (May '38). The river forecasting work of the Bureau is carried on at 56 centers throughout the country from which forecasts and warnings are issued to nearly 800 communities. On lower reaches of the principal rivers the Bureau has for many years accurately forecast and published, days and even weeks in advance, the approach of flood crests. Our nation is rapidly becoming flood conscious with result that the Weather Bureau has been forced to extend its river forecasting service into the headwaters areas of nearly all the principal streams. At many important up-river stations the forecaster has only a comparatively few hours to foretell the approach of a flood. Pittsburgh is perhaps best example of such a situation. To provide the forecaster with means of estimating stream flow from rainfall the country has been divided into nine regions to coördinate and unify data for this purpose. Gathering of evaporation data has been a responsibility of the Weather Bureau for many years and it maintains nearly 60 class A evaporation stations.—*H. E. Babbitt.*

River Measurement and Water Studies. JOHN C. HOYT. Eng. News-Rec. **121**: 269 (Sep. 1, '38). Collection of hydrological and meteorological data in U. S. is reviewed from its inception. Development of most sections of country has progressed to stage where difficulty is experienced in obtaining supply of water to meet all public requirements, resulting in widespread appreciation of need for definite facts on water. On Oct. 31, '37, there were in operation 3,381 river measurement stations, about 75% being equipped with water-stage recorders. Including those from abandoned stations, records are available for about 7,400 stream gaging points throughout country.—*R. E. Thompson.*

Vagaries of Run-Off from Catchment Areas. ANON. Surveyor (Br.) **93**: 722 (May 27, '38). *Report of paper by* CALEB MILLS SAVILLE *before Inst. of Water Engineers.* Unfortunately "normal" rainfall is never experienced, the spread between max. and min. varying widely between seasons and between adjacent watersheds in same season. For example, a combination of unusual conditions resulted in a great flood at Hartford, Conn. in '36. Chances of repetition of

the condition are about 1:330. Total run-off in the storm passing Hartford equivalent to 31″ on the Thames basin above Teddington Weir.—*H. E. Babbitt.*

The Dartmoor Catchments. R. Hansford Worth. Surveyor (Br.) **93**: 860 (Jun. 24, '38). *From paper before British Waterworks Assoc.* Of general interest because of effect of geological and physical conditions of the watershed on run-off. Dartmoor is a granite highland, 248 sq. mi., in which there is only negligible vestige of sedimentary rocks. Surface gradients are varied, rivers and streams radiating in every direction. The product of weathering of the granite rock is known as "growan," a sandy mass, strong enough to support itself at an exposed face, but yielding to the pick. Growan has considerable pore space. The soil of Dartmoor is shallow and peaty, normally holding 85% moisture. Loses its water with difficulty. Water which has penetrated the peat is thus protected from evaporation by this wet blanket. The pore space of growan is thus supplied with water, which flows slowly through it, providing a reserve for streams during drought. Mean annual rainfall, omitting the eastern division, ranges from 57″ to 83″. In computing run-off no factor of evaporation should be applied. A prolonged spell of wet weather makes toward a full return of rainfall as flow-off, and fills the storage of the ground. Areas ranging from 3,620 acres to 400 acres have run-offs from 0.6 to 0.8 c.f.s. per thousand acres, while in flood time an 800-acre watershed has shown a flow of 370 sec. ft. per thousand acres. A rainfall of 3″ in 30 min. is a possibility. Under such conditions flows of 6,000 sec. ft. per thousand acres might occur. Actual floods are evanescent. Catastrophic flooding has been reported.—*H. E. Babbitt.*

Man, Land and Water in East Africa. C. E. Gillman. Afr. agric. J. **3**: 329 ('38). Discusses the cycle of rainfall, run-off and evaporation with special reference to conditions in East Africa. Sufficient data are available to give rough idea of general distribution of rainfall but more widespread measurements are needed. There is little information concerning variability of rainfall and the minimum to be expected. In some areas evaporation may be equivalent to depth of 2 cm. per day at certain times of year. Relations between run-off, absorption by the soil, and transpiration by plants are discussed. The conservation of water includes protection of vegetation on watersheds, prevention of land-erosion in the middle reaches, and training of rivers in the lower reaches. Storage of water has many difficulties such as high evaporation and large amounts of silt. Tanks to store water collected from a roof are used for European dwellings, but native roofs are unsuitable. Ground water is usually protected from losses by evaporation, but water at levels below 30 meters is of little use to native communities. Irrigation is considered and it is concluded that only minor projects are suitable for East Africa.—*W. P. R.*

WELLS AND GROUND WATER

Ground Water Pollution Studies. J. Infect. Dis. Ground water (g.w.) pollution (pol.) from experimentally bored hole and pit latrines, in sandy soil,

has been studied by the Alabama State Dept. of Health. The reports cover certain important phases of the problem and reflect careful work. In the first article,—*Ground Water Pollution and the Bored Hole Latrine.* E. L. CALDWELL AND L. W. PARR, *Ibid.* **61**: 148 (Sep.-Oct. '37) (see also abstract J. A. W. W. A. 30: 710 (Apr. '38)), preliminary studies demonstrated that typical *Esch. coli* and chemical (chem.) products were obtained only from wells in or near sewage flow from the latrine, traveling with the g.w. Such wells, yielding chem. pol. but no fecal organisms, emphasized the importance of "defense" processes restricting flow from latrine and filtration bed by clogging of interspaces with organic aggregates. Gas production in 13% of the samples from wells outside the flow was "attributed to *Aer. aerogenes* and *Cl. welchii* from infected pumps or surface percolation from rains". In the main investigation findings indicated latrine was essentially a septic tank in immediate contact with filter bed (sandy soil). Effluent, containing fecal organisms and particles in suspension, also chem. products in sol., issued into all strata subtended and was carried with the g.w. flow toward the drainage channel, forming a pol. stream distinguishable from the normal g.w. Lineal and depth recovery of pol., in relation to latrine conditions, brought out that (a) initial advance is a function of velocity and concentration. In lower strata, *Esch. coli* were detected first and chem. pol. later. In upper strata it was reversed; (b) clogging in the filtration bed, increasing sludge and scum mat formation, and greater viscosity of intermediate fluid results in gradual regression of the bacterial (bact.) stream and diminution of chem. characteristics; (c) rise in g.w. above the packed sludge results in a new pol. flow through sands not previously infiltrated; (d) "in the early stream typical *Esch. coli* at first predominated but were shortly accompanied by *Cl. welchii* which preceded the appearance of *Esch. coli* at distances beyond the initial and were recovered long after disappearance of colon organisms during regression. As the stream aged and regressed, atypical *Esch. coli* and other colon forms, typical and atypical, accompanied typical *Esch. coli* or were the only forms recoverable"; (e) stream pol. characteristics at varying depths were complexly related to velocity and strata position, in relation to latrine conditions. In general the greatest conc. of chem. and bact. characteristics, in the period of significant flow, was in the middle stratum. Direction of latrine flow varied with the g.w. flow and resulted in expansion of the pol. stream. Expansion of colon stream was transient while spread of *Cl. welchii* was more prolonged. Chem. dispersion was greater than the bact., yet the anaerobic stream width approximated that of the chem. Presence of fecal organisms generally coincided with max. chem. concs. The desirability of limiting the lineal travel of coliform organisms is brought out in the—*Study of an Envelope Pit Privy.* E. L. CALDWELL. *Ibid.* **61**: 265 (Nov.-Dec. '37). It demonstrates the marked influence of g.w. flow velocity upon this lineal travel and suggests a practical application in safeguarding the g.w. from dangerous contamination in areas of suspected or observed high velocities by interposition of an envelope of fine textured soils around the latrine and reaching into the g.w. When the general direction of flow is known, only the lower (down stream) half of the pit would require protection. During the experiment, test wells 10′ away showed "complete absence of *Esch. coli* and practical absence of any colon-aerogenes

organisms". On the other hand, chem. pol. was present. The findings also suggest some restriction of contamination with anaerobes of the *Cl. welchii* type. Influence of an impervious stratum on pol. flow in g.w. immediately above is considered in the next article,—*Pollution Flow From Pit Latrines When an Impervious Stratum Closely Underlies the Flow.* E. L. CALDWELL. *Ibid.* **61**: 270 (Nov.-Dec. '37). The impervious stratum into which the solid end-points of test wells were sunk, consisted of dense calcareous material. Author concludes that generally no essential differences exist in principles of pol. flow from different type latrines reaching into g.w. immediately above the stratum controlling its motion. Any differences noted are of degree rather than kind. Rate of flow predominantly determines the lineal travel of fecal organisms. Related also are the conditions controlling time and degree of flow. Soil conditions, g.w. levels and type of construction affect outflow. In a small diameter bored latrine, sludge piles up rapidly and scum forms soon at the top to block outflow. Greater vol. per depth of penetration in a pit latrine results in more prolonged flow and greater digestion. Under high velocity flow, increasing in rate from discharge outlet and being sustained through good transmitting sands during all changes in water levels associated with prolonged fluidity of latrine content, significant numbers of *Esch. coli* were carried 80′. Gross pol. existed in mid-flow between 40′ and 60′. At end of experiment (16 mos.), due to "defense", the *Esch. coli* stream had regressed between 40′ and 60′ where only an occasional one was recovered. At 20′ where the colon-aerogenes index indicated more than 100 per cc. the first few mos., *Esch. coli* was restricted to 10 cc. vols. In contrast, consideration is next given to the effect of permeable subsoils on g.w. pol. in a report on—*Pollution Flow from a Pit Latrine when Permeable Soils of Considerable Depth Exist Below the Pit.* E. L. CALDWELL. *Ibid.* **62**: 225 (May-Jun. '38). This investigation was made under conditions radically different from those in previous studies. Pit privy (3′ x 3′2″ x 8′8″ deep) was located in an experimentally controlled field in the terrace plain of a drainage stream flowing at considerable distance away. Water-table slope was only a "few inches per 100". Pit was in fine to medium sands with flow rates of 2″–6″ per day. Between pit floor and impervious stratum were 7′–8′ of medium to gravelly sands with flow rates of ½′–2′ per day. Excreta from a large family were added daily. Initial depth of material in the pit was 6″ which later rose to a max. of 5.47′. In general, the study not only further confirmed principles of pol. flow already indicated but also demonstrated a new principle, i.e., "a pol. stream describes a path, the resultant of the interacting forces of lineal flow with the g.w. toward the discharge stream and variations in densities of effluent flow in comparison to the g.w.". The chem. pol. was traced between 325′–350′ by odors and pH, and in lineal extent for 310′ by chem. analyses. These demonstrated strikingly the variations in intensities with distances from source, and existence of gross chem. pol. at 80′ plus. Owing to dispersion of lag during oscillation of stream with changes in g.w. motion, stream width expanded from 3′ in 3 mos. to a max. of 25′ (at 80′ distance) in shifting over an approx. expanse of 50′. Initially the stream flowed with the g.w. through a 6″ depth but shortly dipped below the pit floor plane. It increased to 3½′ in 3 mos. and to 7′ in 6 mos. In lineal section it described a curvilinear path for a ways because

of its greater density, then flattened slowly with upper and lower limits converging to zero. The action of the *Esch. coli* stream is of considerable interest. It formed an inner core of less width and depth and markedly less extent than the chem. stream. Initially *Esch. coli* advanced slightly beyond 10′ in 3-4 mos. when regression factors were less active. At termination the apex barely reached 5′. The *Esch. coli* stream expanded from 1½′ in 3 mos. to max. of 4′, for a brief time, at 5′ (lineal) distance after 6 mos., then rapidly contracted to less than 1′. Its depth increased to max. 3′ in 6-7 mos. At the end it had contracted to 6″ or less. Expanse in width and depth was at expense of lineal concs. Max. *Esch. coli* recovery in 3-4 mos. approximated 200 per cc. at 5′ while at the close were present only in 10 cc. Conc. of colon organisms, directly below pit at the close, was less than the number found 1′ away in the early stream. Coliforms other than *Esch. coli* and atypical *Esch. coli* comprised largely the surviving colon organisms. Anaerobes of the *Cl. welchii* type probably reached 50′ in significant conc. Rate of g.w. flow is considered in the paper,—*Direct Measurement of the Rate of Ground Water Flow in Pollution Studies.* E. L. CALDWELL AND L. W. PARR. *Ibid.* **62**: 259 (May-Jun. '38). Latrines were used as charging sources of common salt and ammonium chloride. In general, salt sol. leaves the charging source gradually. Apex of flow is subject to dil. with g.w. and with dispersion in depth, as in transverse section, greater dil. occurring at the margins. Subsequent outflow is less subject to dil. but through continued dispersion, marginal limits expand and the salt sol. is constantly diluted until no longer distinguishable from the g.w. Thus, the foregoing papers tell us that when human excreta is deposited in sub soil, in the zone of saturation, the general course and approx. distance intestinal bacteria will penetrate unconsolidated soils with g.w. flow is predictable in any particular instance. In the zone of saturation fecal organisms travel only lineally with the g.w., the depth and width of recovery being governed by the form and motions of the pol. stream. Of themselves they do not migrate up-stream or laterally. In subsoil, they remain localized except as mechanically transported by other agencies. However, deposition of human excreta in subsoil above the zone of penetration involves a different set of relations, according to the report,—*Studies of Subsoil Pollution in Relation to Possible Contamination of the Ground Water From Human Excreta Deposited in Experimental Latrines.* E. L. CALDWELL. *Ibid.* **62**: 272 (May-Jun. '38). This work shows that intestinal organisms are confined to the depositary except as mechanically carried downward with gravity flow if the soil is pervious. Lateral transfer is normally insignificant unless retarded percolation causes impounding. The area of saturation then is dome-shaped. Significant correlation existed between moisture conditions and pol. from excreta deposits into the subsoil. When the water table in general was 1½′-2′ below exp. pit floor and within 6″ for 1 mo., contamination did not appear in test wells 5′ away in line of g.w. flow. With latrines located on a cliff in an area of comparatively low g.w., colon organisms did not penetrate 1′ below or 1′ laterally from a dry pit latrine. From one subject to seepage from rains, colon organisms were carried 3′ but not 4′ below and 1′ laterally, but not in significant numbers. From one to which 100 gals. water were added daily, organisms traveled 6′ but not 7′ below and about 2′ laterally, though in significant numbers only 1′. The limitation of pol. is due to rapid "defense"—clogging of pore spaces with fine

fecal particles and aggregates of colloids and organisms—which, added to accumulated sludge, retards flow and thus "increases the time for the death rate of fecal organisms to operate". This highly informative series, 129 pages, is well supplemented with photographs, diagrams, figures, maps and tables.—*Ralph E. Noble.*

Control of Quality of Water Supplies. F. DIENERT. Technique Sanitaire (Fr.) **32**: 29 (Feb. '37). Urges the great importance of the sanitary survey in the field of any proposed source of ground water supply, from which, when a water originally pure gradually becomes polluted, one may generally deduce the reason. A single analysis has only limited value; as it can only attest to conditions at the time sample is taken, it may prove quite deceptive. The hydrological aspects of the geologist's report are very important, above all, the filtering capacity of the gathering grounds; it should be stated whether the water gets turbid in winter and whether its flow is constant,—the significant indications of access of surface run-off. The nature of the gathering ground is also important, whether inhabited or forested, and if inhabited, nature of sewage disposal. If forested, some turbidity in winter is not of much importance, but if inhabited, undesirable pollution is to be expected and repeated analysis must be made to determine the filtering capacity of the soil. As this is expensive, it is often not done, notwithstanding that it is usually specified that a well or a spring should be observed for a full year before being adopted as a source of supply. Frequent analysis during this period is very advisable. However, it must be borne in mind that it is only at times of heavy rain that analysis can detect gross pollution, indicating the lack of filtering capacity of the soil. For example, the author once experimented with *Esch. Coli* (acclimatized for differentiation purposes to arsenic) in a cess pool receiving constant flow of 2 liters per sec. He recovered these organisms in a well which was previously known to be subject to infection from the cess pool; but to do so he had to use liter samples, the organisms were so scarce. Yet the terrain was badly fissured and circulation of subsoil water active. Geology, supplemented by fluorescein test, would have furnished all the data needed in this case. After supply once has been inaugurated, the quality of the water must be maintained, and for this frequent analysis is of little use, if the source has been properly surveyed. Either the terrain has or has not adequate filtration capacity, giving clear water at all times and a nearly constant flow (variation 1 or 2%). If it has, then only the immediate neighborhood of the spring needs attention. It should be closely examined with a very thorough knowledge of the various ways in which contamination can reach a well or spring. If this examination yields nothing, neither can bacteriological examination add anything. However, a close watch is needed on the gathering grounds, that no septic tank be installed. If the land is forested this is not possible, but if the terrain be fissured and not filtering, there is always risk of pollution, and analysis, unless carried out daily, is of slight use to ward off this danger promptly. Sterilization is the proper course in such cases, and analysis is of use only to verify its efficiency.—*Frank Hannan.*

Formulas for Underground Flow. EDOUARD LEFRANC. Génie civil **113**: 166 (Aug. 20, '38). Darcy's law was developed for vertical filtration and ex-

tended by Dupuit to horizontal underground flow. But Dupuit used a vertical surface, whereas actually surfaces of equal potential should be used, that is surfaces that lie at any point at right angle to the stream-lines. The concept of the resistance to the flow of underground water is developed by subdividing the whole body of flowing water by surfaces of equal pressure; s being the area of any one of these surfaces, k the av. coef. of permeability of the material between two surfaces, and l the average distance between them along the lines of flow. Then, letting l approach the limit 0 the resistance $R = \Sigma \frac{l}{ks}$. This new law for underground flow is stated as follows: "In any permeable massif through which flows an underground stream and which massif or part thereof is bounded upstream and downstream by surfaces of equal pressure and at the sides by continuous stream-lines, the difference in potential of the two surfaces of equal pressure is the product of the resistance of the massif and its outflow." This law applies to all types of flow through permeable soil. It is similar to Ohm's law in electricity and allows the same type of calculations. Applications of the method are given and experimental data are presented to show that Darcy's and Lefranc's law are of the same exactness of most laws and formulas used by engineers and therefore should be used as the basis in formulating a precise science of underground flow.—*Max Suter.*

The Movement of Water in the Soil. A. Vibert. Génie civil (Fr.) **113**: 7 (Jul. 2, '38). Based on Darcy's law formulas are developed that avoid the influence of two simplifying hypotheses made in the derivation of Dupuit's classical formula, namely the assumption of a uniform velocity distribution in a vertical element and the replacement of $\frac{dy}{ds}$ by $\frac{dy}{dx}$ for necessity in the integration. The influence of these simplifications is especially great in cases of high draw down with correspondingly steep hydraulic gradients. Two cases of ordinary underground flow are considered: (1) outflow along a gallery or river and (2) outflow into a well. In both cases the derivation is based on the same principle. The flow between two very close elements in the direction of flow is considered. The constant velocity from Darcy's law $v = Ks$ is given the inclined direction towards the intersection of the tangent to the surface of the ground water with the bottom of the ground water stream. For the outflow per unit length along a line it is then found

$$Q = K\ p\ y \text{ arc tg} \frac{dy}{dx} = K\ p\ H\ \theta$$

and the inflow into a well is calculated to

$$Q = 2\pi K\ p\ x\ y \text{ arc tg} \frac{dy}{dx} = 2\pi K\ p\ R\ H\ \theta$$

where K is a factor of permeability depending on the soil, p is the ratio of the open area to the full area of the water-bearing strata, dx is measured in the direction of horizontal flow, H, and y is the total depth of water at the place where θ is the slope of the surface of the ground water expressed in radians and

R is the distance of this section from the center of the well. The use of these formulas in wells with large drawdown gave very reasonable results in the calculation of permeabilities.—*Max Suter.*

On the Influence of the Type of Construction of Shallow Wells on the Quality of the Water Drawn. MILIVOJ PETRIK. Gas-u. Wasser. **81**: 413 (May 28, '38). As an introduction of sanitation in Jugoslavian villages, the Institute of Hygiene at Zagreb selected the village of Mraclin (pop. about 1100) as object of study and demonstration. Village was sewered and the water supply improved by construction of twelve sanitary wells. Water is obtained from gravels that lie to a depth of 30′ under clay cover 5′ to 8′ thick. The old water supply consisted of many wells of several types of construction, all with open tops and mostly only about 20′ deep. The new wells were dug to the bottom of the gravel, shaft and top built with water-tight concrete and equipped with Caruelle pumps. Water from some of the old open wells and from some of the new sanitary wells was then tested weekly for a period of over one year for chemical composition and bacteriological content. The results of these tests are given and discussed in detail. They show clearly that the open wells are polluted by sewage and growth, whereas the water of the new wells is much better in every respect, although it is not bacteriologically perfect. The fact that the water of the new wells comes from the same water-bearing strata yet from a lower level than that of the old shallow wells has a great influence in improving the quality.—*Max Suter.*

The Protection of Underground Sources of Water Supply. EDGAR MORTON. Surveyor (Br.) **93**: 858 (Jun. 24, '38). *Extracts from paper before Br. Waterworks Assn.* Beneficial governmental control of competitive pumping of underground supplies has been exercised but is seriously inadequate and there are many inequalities in legislation. Case at Wolverhampton affords exceptional example of hardship suffered by a local authority, where the supply has been reduced 50% through industrial plants within a radius of two mi. Utilization of underground supplies is bristling with hydro-geological difficulties through uncertainties as to quantity and quality. Arrangements should be made to maintain a record of frequent and continuous observations of water levels in private wells within convenient radius of large pumping centers. Always problematical to what extent protection against pollution should be afforded in a sandstone area. Protection of surface watersheds may be afforded through: (a) purchase of the area, (b) creation of by-laws controlling surface pollution, (c) remedial works to abate pollution, under court order, and at the expense of the water works authority. Has been found inexpedient, in order to protect water supplies, to restrict planning schemes without compensation to the land owner.—*H. E. Babbitt.*

Variations in Composition of Sub-surface Water in the Swakop River, South-west Africa. T. W. GEVERS AND J. P. V.D. WESTHUYZEN. S. Afr. J. Sci. **33**: 231 ('37). The Swakop River is 230 mi. long and drains the interior uplands of Central South West Africa and for the last 100 mi. of its course flows through the Namib desert. The annual rainfall near the source is 15–16″, but 100 mi.

to the west it is only 7-8", and in the coastal belt there is practically no rain. The bed of the river is usually dry but sub-surface water is abundant at a shallow depth. Samples of sub-surface water from 36 positions between the source and the mouth were analysed. Near the source the waters are soft and pure and the salinity gradually increases as the coast is approached. Up to the edge of the Namib desert calcium carbonate is the preponderating salt. Sodium, magnesium, sulphate and chloride gradually increase, and in the last 100 mi. sodium chloride is greatly in excess of any other salt. The total dissolved solids vary between 8.8 parts per 100,000 at the source to 673.5 parts per 100,000 near the mouth. The waters are compared with those from boreholes in the interior and with those from various rivers.—*W. P. R.*

Content of Heavy Water in the Entrails of the Earth at a Depth of 1300 Meters. L. M. Shamovskiĭ and N. F. Kapustinskaya. Acta Physicochim. U. S. S. R. **7**: 797 ('37) (in English). Samples of water taken at depths of over 1200 m. from drill holes in Moscow region were investigated. Flotation method was used for detn. of D_2O content. Results indicate that in deep layers of earth's crust there is an increase in D_2O in H_2O; $D_2O:H_2O = 1:4490$.—*C. A.*

PUMPS AND POWER EQUIPMENT

The Use of Gas as a Boiler Fuel. O. A. Bowen. J. West. Soc. Engrs. **43**: 43 (Feb. '38). Some of principles governing application of gas to common steam boilers are given. For brick set boilers multi-tube and register type burners most popular, arranged for induced or forced draft. Combustion volume should be ample, from 0.8 to 2.5 cu. ft. per developed hp. required. Multi-tube burner especially well adapted for operation with 2 to 16 oz. gas pressure; register type with 2 lbs. or more pressure. Total or partial premix types of burners used in firebox type boilers. In total premix burner all air for combustion mixed with gas before admittance to furnace. Burns with short flame and operates very well in combustion chambers of less than ½ cu. ft. per developed hp. Area of coal designed stack will be amply large for equal loads of gas or oil, usually can be depended on to carry some overload. Possible to arrange for both gas and oil burning in same installation. Efficiencies over 70% obtained with gas in boilers not equipped with economizers or air preheaters. Corresponds approx. to 14.5 therms of gas req'd per 1000 lbs. of steam at 100 lbs. pressure, 180° feed water temp. Suitable equipment can be obtained to give simple installation, low in maintenance, high in efficiency and capable of producing low cost steam throughout its useful life.—*Martin E. Flentje.*

Some Experiences in the Use of Scale Models in General Engineering. Richard W. Allen. Engineering (Br.) **146**: 243 (Aug. 26 '38). Scale models now widely used in general engineering preliminary to design. It is impossible to obtain by calculation the performance of a centrifugal pump in any way but from test measurements. There is only one escape from this burden,—through measurements which can be taken from models more conveniently, economically and rapidly than in tests of final product. Quantitative information from models must be interpreted in accordance with laws of dynamic simi-

larity. Technique recently developed for testing of water pumps by using air as a working fluid, by which it is possible to find for a centrifugal pump: duty, characteristics, running speed, and other qualifying conditions. If the test proves unsatisfactory it is possible through the model to find: shapes or relative proportions of elements of the pump and to give the specified characteristics. For example, four pumps of different duties were designed by model tests, duties varying between 10 m.g.d. (Imp.) against 150′ to 28 m.g.d. (Imp.) against 280′. Models made of aluminum and wood to scale of $\frac{1}{5}$ and were operated with air as a fluid. Two other models made to $\frac{2}{5}$ scale for corroborative information.—*H. E. Babbitt.*

Corrosion—Cause in Pumps. C. H. S. TUPHOLME. Chem. Industries. **42:** 159 (Feb. '38). Most corrosion cases in centrifugal pumps and hydraulic turbines are due to combined chemical and mechanical action. Water contains considerable air and other gases. Small air bubbles are released due to high suction head and/or eddy currents and adhere firmly to the material. The oxygen and carbon dioxide in these bubbles react with the metal and the intermittent washing with water at high speed removes the oxide providing a more favorable point for corrosion. This type of corrosion is limited as a rule to pumps to which water does not flow under pressure. The most effective means of eliminating it is to fit an air chamber with air separator before the pump, the air liberated being removed by an ejector or air pump. Otherwise the pump should be installed, when possible, at such a level that the water enters under pressure. Less common, but as dangerous, is corrosion by electrolytic action in pumps handling acid or salt water. In such cases any possibility for galvanic action should be entirely eliminated.—*T. E. Larson.*

Welding Impeller-Vane Tips. DANIEL MCFARLAND. Eng. News-Rec. **120:** 344 (Mar. 3, '38). At Grand Coulee Dam, abrasive sand and cement in concrete-cleanup water soon wears out impellers of 80 small sump pumps at ends of vanes. Salvage effected by building up vane with manganese-bronze welding rod and then finishing it off smooth. To avoid difficult task of finishing inside of curve, V. Esser developed template of graphite or hard carbon which is held in place by using back of next vane for fulcrum. Vane is built up and excess metal cut off in lathe or by using a file as cutting tool as pump is turning. Cost is about $2 compared to new cost of $18, and rebuilt ones, being a little harder, sometimes outlast new ones.—*R. E. Thompson.*

The Pollution of Air by Diesel Engines. MAURIN AND ANDRÉ KLING. Rev. pétrolifère **768:** 49 ('38). The CO content in the exhaust gas of Diesel engines is very low (0.32–0.5%), at most 1.5% (intentional maladjustments of the engine). The amt. of smoke and its odor are less at full load than under part load. Unburnt solids and liquids are always present in the exhaust, being in the form of lubricating oil, org. acids, aldehydes and oxides of S. Purification by means of a water bath and cyclone-type separator in series is capable of eliminating 220 g. (79%) out of a total of 280 g. of solids and liquids, the other 21% being removed with an activated-carbon

filter. Addn. of Na_2CO_3 or $Ca(OH)_2$ to the water bath is recommended.—*C. A.* Editor's Note: While the danger of carbon monoxide poisoning is not great from the exhaust of a well operated full Diesel engine, nevertheless, caution must be exercised to avoid asphyxiation due to the extreme deficiency of oxygen.

Insurance of Electric Motors. Anon. W. W. Inf. Exch., Canadian Sect., A.W.W.A. **2**: C:3: 15 (Dec. '37). Of 78 Canadian waterworks listed, only 14 carry insurance against failure of electric motors. Annual premiums paid total $3,743.80, while total loss covered by insurance during 10-yr. period was $5,777.08 (in 10 different municipalities).—*R. E. Thompson.*

To Check Pump Hum in Pipes. Anon. Eng. News-Rec. **120**: 590 (Apr. 21, '38). Substitution of 60- for 50-cycle motors in Pasadena resulted in vibrations which caused complaints. Investigation showed that hum was transmitted through water column and not through metal of pipe system. Insertion of surge chamber made of 18–24" section of 4- or 6-in. pipe in line on service side of meter was found to eliminate hum. Meter noises are also eliminated.—*R. E. Thompson.*

STEAM PLANT CORROSION

Chromates for Corrosion Prevention in Standby Boilers. R. C. Ulmer and J. M. Decker. Combustion **10**: 3:31 (Sep. '38). Brief review is given of several methods of preventing corrosion in standby boilers. Use of uniformly distributed alkaline chromate solution is described. A 30–50 p.p.m. hydroxide concentration and 100–200 p.p.m. sodium chromate concentration is maintained. Appreciable chloride concentration was found to necessitate greater chromate concentration.—*T. E. Larson.*

Caustic Embrittlement—Causes of and Recommended Methods of Prevention. C. L. Crockett. Ann. Rept. Ohio Conf. Water Purif. **17**: 40 ('37). A resume of work of leading investigators. Embrittlement cracks follow grain boundaries and are known as intercrystalline or intergranular cracks. Other two types of cracks, due to corrosion and fatigue, cut through grains in their progression. This is illustrated by micrographs. Although embrittlement has been attributed to other factors, e.g., hydrogen, most generally accepted theory is that it is due to concentration of sodium hydroxide around rivet heads, etc., with steel stressed beyond yield point. Experiments have shown that definite amount of silica must also be present. Accepted method of prevention is addition of sodium sulfate to embrittling water, in amount sufficient to maintain definite ratio of sodium sulfate to sodium carbonate, and elimination of crevices in which caustic material may concentrate. Curve (reproduced) has been developed which will show at glance, from analytical data, whether water is embrittling. Experiments are in progress on use of lignin as preventive. *Discussion.* R. C. Bardwell. *Ibid.* 51. Caustic embrittlement is not the correct term for the cracking. It is intergranular corrosion and may or may not be due to caustic. Caustic soda by itself will not cause intercrystalline cracking of steel, under stress or otherwise: silica, lead oxide or other substances must also be present. Other combinations of compounds

will cause intergranular corrosion, e.g., sodium nitrate and manganous chloride or nitric acid and manganous chloride. To prevent intergranular corrosion, film must be maintained over metal and grain boundaries. Experiments and experience with locomotive boilers have shown that sodium sulfate is ineffective under some conditions. Phlobo tannins appear to offer definite protection, within certain limits. Lignin compounds have proved effective. Paper mill waste (sulfite cellulose slops) has been found to be one of most effective materials for protection against intercrystalline corrosion up to 500 lbs. pressure.—*R. E. Thompson.*

Bureau of Mines Investigation on the Intercrystalline Cracking of Boiler Steel. W. C. Schroeder, A. A. Berk and R. A. O'Brien. Bul. Am. Ry. Eng. Assn. **404**: 76 (Jul. '38). Quality of boiler water is only one of four factors entering into cracking of stressed boiler steel by intercrystalline corrosion, the others being (2) stress or cold work in the metal, (3) concentration of the boiler water, and (4) contact of the concentrated boiler water with stressed or cold worked metal. Intercrystalline corrosion in boiler steel highly stressed either by external applied load or internal cold work, will result from exposure of such stressed metal to solutions containing from 75,000 to 500,000 p.p.m. of NaOH with a slight amount of SiO_2. The average boiler water concentration does not exceed 1000 p.p.m. NaOH, therefore small leaks must exist which permit the further concentration of the boiler water in the capillary spaces between seams adjacent to the stressed metal. The laboratory work does not show that Na_2SO_4 will protect the steel against intercrystalline cracking and NaCl is a slight accelerating agent. Further inorganic salts tested with NaOH-SiO_2 solutions at high temperatures were not effective. The laboratory work shows that cutch, quebracho and waste sulfite liquor are effective in preventing intercrystalline corrosion regardless of NaOH content, if added in proper amounts to stressed specimens at temperatures equivalent to boilers operating below 500 lbs. pressure. This is in line with the experience on one railroad which had used such material for over 10 yrs. and practically eliminated this type of embrittlement. Attention is called to the advisability that steps be taken in boiler construction to reduce cold work stresses all possible, and eliminate leakage. Treatment of the water with cutch, quebracho, or waste sulfite liquor is an added precaution. Photographs, charts, and illustrations are presented.—*R. C. Bardwell.*

Cause of and Remedy for Pitting and Corrosion of Locomotive Boiler Tubes and Sheets with Special Reference to Status of Embrittlement Investigation. R. E. Coughlan et al. Bul. Am. Ry. Eng. Assn. **404**: 73 (Jul. '38). With use of larger engines, increased boiler pressure, longer engine runs, and greater utilization of the power, a type of cracking has developed in occasional engines starting from rivet holes in the stressed metal either in boiler shell laps or butt straps, which has the appearance of embrittlement caused by intergranular corrosion brought about by chemicals in the water. Maintenance of a sulfate—alkalinity ratio in excess of the A.S.M.E. recommended boiler code has afforded no apparent relief. Satisfactory results on the Chicago & North Western Railroad using lignin compounds in connection with water

treatment over past 14 yrs. appears to confirm laboratory findings of Bureau of Mines research work under Dr. Schroeder to the effect that such organic compounds are much more effective than Na_2SO in preventing intergranular corrosion and their use is being extended on other railroads.—*R. C. Bardwell.*

The Prevention of Boiler Scale by Protective Colloids. N. F. Ermolenko and N. M. Zhuromskaya. J. Applied Chem. (U. S. S. R.) **10**: 2009 ('37). Artificially prepd. waters of 8.8–46.3 German degrees of hardness were used with colloids in 0.01, 0.05 and 0.2% concns. The antiscaling action decreases in the order: tannin, agar-agar, starch and gelatin. The mechanism is explained by the ability of the protective colloids to stabilize the ultramicrocrystals formed and to retain these crystals in soln. as colloids. Four references.—*C. A.*

Electrolytic Protection of Steam Power Equipment. H. Bendfeldt. Arch. Wärmewirt. **19**: 123 ('38). Condenser with wrought iron header and brass tubes was protected from corrosion by connecting tubes (cathode) to d.c. of about 6 volts and 2.5 amp. Similar installation on evaporator produced a very loose scale on tubes. Inserted C or Fe anode is insulated from condenser or evaporator.—*C. A.*

A Case of Corrosion Caused by Concentration Cells. H. Grubitsch. Korrosion u. Metallschutz **14**: 113 ('38). From distribution of pits on bottom plate of boiler feed water tank receiving water of 2 different compns. and absence of irregularities in steel itself, it is concluded that corrosion was due to concn. cells. Max. attack occurred between the 2 water inlets.—*C. A.*

DAM DESIGN AND CONSTRUCTION

The Design of Rock Fill Dams. (*Discussion of previous paper,* see abstract J.A.W.W.A. **30**: 379 (Feb. '38)). Proc. A.S.C.E. **64**: 573 (Mar. '38). Howard F. Peckworth. A rock-fill dam with impervious, rolled-earth blanket should come under definition of rock-fill dams. Oren Reed. Features of the more important of the few rock-fill dams are presented. A well-designed rock-fill dam is relatively stable structure and in cold locations rock-fill may be preferred to concrete. Walter L. Huber. Many variations have been developed from author's type of dam, including important structures built in recent years in western U. S. Advantage secured from wood facing is flexibility allowing it to follow unequal settlement and remain comparatively water-tight. Disadvantage is fire risk when dry. Samuel B. Morris. Behavior of rock-fill dam during earthquake is questionable since fills usually settle and crack during earthquake. In case of high rock-fill settlement as much as 5% expected under normal conditions due to spalling of the rock under load. Settlement may continue for months or years and offers serious menace to safety in an earthquake. L. F. Harza. Rock-fill dam with concrete face will inevitably require repair of face after initial settlement. *Ibid.* **64**: 837 (Apr. '38). Paul Baumann. Measurements of shrinkage of San Gabriel dam during construction indicate increase in density from 100 to 125 lb. per cu. ft. O. W. Peterson. Unsegregated quarry-run mass, with excess fines

wasted, will give highest degree of contact, rock to rock, and greatest fill density. Such fill will have ample rock-to-rock contact and still sufficient voids for drainage. Volume of water supplied, rather than pressure, emphasized as of importance in reducing frictional resistance to movement of rocks against each other and in causing fines to lodge in spaces. No reason for delaying a subsequent lift after previous one sufficiently completed for safety to men and equipment at lower levels. Up and downstream faces of each lift should approximate required final positions of the loose fills. Gates, of generous proportions, may be installed on spillway lip of rock-fill dam. This presupposes necessary vigilance in operation. GEORGE W. HOWSON. During const. of rock-fill dam three phases generally simultaneous,—placing fill, laying rubble section of upstream slope, and pouring of concrete ribs and apron. The larger the rock in fill the fewer will be the bearing points for any given depth of fill resulting in less settlement, but const. equipment and methods generally limit size of rock. Large rocks generally least damaged in quarrying and have soundest faces, and least points upon which rock-to-rock load is carried. Beneficial effect of sluicing great. Considerable settlement in lower parts of dam during const. as latter assume increased weight of fill. At Dix Dam found advisable to keep concrete ribs about 50′ below top of rubble wall. No standard design of facing claimed best for all conditions. If concrete facing adopted should be poured directly against dam and between the face rock, with expansion joints. Vertical joints constructed open or closed according to position in dam. *Ibid.* 64: 937 (May '38). JOHN E. FIELD. Steepest slope permissible in dam design is that at which movement will start, but this has not been determined by research or experiment. Hence, no justification in adoption of the minimum slope of 1:1.3 for rock-fill. In a rock-fill dam presence of erodible material must be avoided. This has caused failures. Core walls in center of dam inadvisable. Gates, and collapsible stop-planks, used in spillways but generally undesirable. JOHN H. WILSON. Attention of designing engineers directed to method for const. of sandstone rock-fill dam involving sluicing of fines into larger rocks to produce more dense fill. FREDERICK H. FOWLER. Author's definition does not include many important rock-fill types developed elsewhere than in the Sierra Nevada of Calif. Even in latter there has been rapid evolution and in later dams placed-rock sections far thinner in relation to heights than in the earlier Relief Dam. Chilean and Algerian practice have migrated to other regions and, though not conforming to author's definition, cannot be placed under other classification. "Chilean type" laminated concrete facing is that used on Cogoti Dam in that country. Similar type facing used initially on San Gabriel No. 2 in Calif., and extremely satisfactory. Difficulties in designing concrete facings warrant consideration of timber. I. C. STEELE AND WALTER DRYER. In design of Salt Springs dam settlement at any point on face of dam estimated to be vector sum of vertical settlement due to the weight of rock in vertical column of rock extending from face to foundation, and movement due to water pressure acting on an inclined column of rock normal to face and having length equal to distance between face and foundation. Arbitrary value given to coefficient of settlement or compaction for this purpose. Face designs were: sliding type, laminated, and monolithic slab. Considera-

tion given to joints, settlement and thickness. Believed that crest settlement at Salt Springs less in proportion to height than any other structure of similar section. F. Knapp. Innovation introduced in Algerian design consists in vibrating rock-fill to reduce voids and diminish settlement. Bakhadda Dam suffered only small settlements and lower slab, with reservoir filled, showed neither cracks nor displacement. Vibrating the fill and sluicing contribute to possible increase of speed of const. *Ibid.* **64**: 1196 (Jun. '38). Ralph J. Reed. Design and const. of rock-fill dam should result in most of settlement taking place before permanent facing is completed. Believed that generous sluicing of fill during const. of San Gabriel No. 2 would have reduced settlement and obviated the failure of concrete face that occurred. F. J. Sanger. Failure of Eildon Dam, rock-fill structure in Australia, began with a slip exposing the core wall. Subsidence continued slowly for some weeks and maximum wall deflection was 4'–8". Cause of failure was probably clay slip in foundation. Remedial measures included adding rock fill to toe, repair of cracks, drainage, and reconstruction of spillway and outlets. C. S. Jarvis. Development of basic idea of rockfill dams may be traced through many centuries. Similarity to rocky bars intruding into streams, and ease of diverting water by brush or litter to close interstices, is striking. Remarkable amount of dry-rubble and other stone-masonry const., backed by rock fill, is found around the Great Falls of the Potomac, attributable to time of George Washington. Advancement in art of dam const. has apparently awarded preference to some form of cohesive integral masonry structure for great storage depths. A structure dependent on loose rock fill for stability seems to qualify as a rock-fill dam whether the impervious element is in form of timber, metal plate, masonry, or dense plastic material used, respectively, as facing or as a diaphragm core, cut-off, or upstream apron. *Ibid.* **64**: 1415 (Sept. '38). L. R. East. The large, rock-fill dam forming the Eildon Reservoir in Australia failed in Apr. '29 due to subsidence of rock-fill on up-stream side of core wall. Concluded that subsidence had arisen directly as result of action of the clay wall; that the clay had acted as a stiff fluid and pushed the rock-fill out into the water; and at the same time, had exerted great pressure down stream, causing the concrete core wall to deflect. Remedies suggested included: restoration of rock-fill by deposition of more rock, underdraining down-stream side of embankment and adding rock-fill to it to increase its supporting power, and repairing cracks and fissures. Francisco Gomez-perez and Miguel Jinich. Low-cost labor and high-cost equipment combined have influenced development of the rock-fill dam in Mexico. Trend has been toward use of smaller rocks than would have been the case if more mechanical equipment could have been used. Most economical types of dam at inaccessible sites are earth and rock-fill. Foundations for rock-fill dams should resist pressure without settling, resist chemical solution and resist erosion. Rock for the dam may be any sound rock. Down stream slope of the Commissions' dams have been 1 1.4, with few exceptions. Waterproof element generally consists of rather thin concrete slab on up stream slope. In exceptional cases steel plate has been used. Design of bottom joint between flexible slab and rigid cut-off wall requires careful consideration. At two Mexican dams the joint has been

made flexible by means of two copper sheets and a layer of asbestos felt. Placing of rock in body of structure by hand in thin horizontal layers reduces settlement considerably.—*H. E. Babbitt.*

Algerian Rockfill Dam Substructures. I. GUTMANN. Eng. News-Rec. **120**: 749 (May 26, '38). Details are given regarding extensive subsurface construction in 4 new dams (Bakhadda, Ghrib, Bu-Hanifia and Fum-el-Gueiss) in Algeria, necessitated by unfavorable ground conditions. Substructures include cutoff walls, grout curtains and elaborate drainage works. Grouting effected with cement, sodium silicate and aluminum sulfate, which react to form aluminum silicate, and, at Bu-Hanifia, by new process known as K.L.M. process. Latter offers following distinct advantages over older methods: (1) Water glass and salt solutions, pre-mixed and pre-treated, are injected simultaneously in one operation. (2) Time of set of injected colloidal mixture can be regulated at will to occur, for example, 1 min., 20 min. or 2 hrs. after preparation. It is claimed that with premature setting and clogging thus eliminated, grout will penetrate into very fine formations and will produce more uniform curtain at lower cost. At Bu-Hanifia, to prevent dangerous regressive erosion under dam by water which might percolate through grout curtains or cutoff wall, protective filter bed of 5 layers of porous materials averaging about 1′ in thickness was constructed under entire rockfill and to line 33′ downstream. Water appearing above 2 lowest layers (sand) will be picked up by drains and discharged into intercepting canal running along toe of dam. Dams are costly structures of placed rockfill made watertight with concrete or asphalt deck slabs laid on upstream slopes.—*R. E. Thompson.* (See also abstract J.A.W.W.A. **30**: 380 (Feb. '38).)

Earthquake Stresses in an Arch Dam. (*Discussion of previous paper*, see abstract J.A.W.W.A. **30**: 379 (Feb. '38).) Proc. A.S.C.E. **64**: 609 (Mar. '38). A. FLORIS. The analysis which takes into consideration only the influence of the inertia forces of the arch, omitting those due to water behind the dam brings up the question as to whether such an analysis can be considered as a criterion of the sesmic resistance of an arch dam. CECIL E. PEARCE. The claim is probably correct that an arch dam, when accelerated upstream, can take an earthquake overload better than any other type because all that results is a slight increase in compression, with no alarming effects in regard to sliding or overturning. However, when the direction of the acceleration is at right angles to the stream, the results are different. *Ibid.* **64**: 989 (May '38). F. W. HANNA. Authors consider only the downstream impulse but the impulse may act either in a downstream or in an upstream direction. Mathematical analysis makes it evident that the algebraic signs of the thrust, moment, and shear are reversed for an earthquake acceleration acting upstream from those of one acting downstream. Steel must be introduced to take the tensile stresses produced. In case of a masonry reservoir effects of earthquake shocks must be applied to the masonry of the structure only; but for full or partly full reservoir the influence of movement of the reservoir water must be considered. Principles used by authors for analysis of thin arches may equally well be applied to thick arches and to arches of variable thickness.

A. W. Fischer. It is doubtful if stresses given by authors for hinged-end condition is the maximum that could occur for an earthquake. Stresses in an arch dam with fixed ends will not be as large as in a two-hinged arch due to the same displacement of the abutments. Ray L. Allin. The cases cited are probably for an arch loaded prior to the earthquake shock with an intensity of compression stress greater than the intensity of tensile stresses indicated by the authors. Where no arch thrust exists prior to earth shock all the reaction at the abutment would be placed where the earth shock is transmitted to the dam and would cause stresses twice those values given by the authors. This case is not so vital to safety as that with full reservoir, yet it may be necessary to stiffen the arch to satisfy the condition.—*H. E. Babbitt.*

Dam Tested by Artificial Earthquakes. Anon. Eng. News-Rec. **121**: 184 (Aug. 11, '38). Morris Dam of Pasadena water system is concrete structure of gravity section, max. height being 328′, height above streambed 245′ and thickness at lowest foundation 280′. Unusual feature is vertical joint near max. cross-section which has several offsets in vertical plane to give sliding contact faces or planes parallel to direction of movement believed to have occurred in old fault which crosses dam site. In cooperation with U. S. Coast and Geodetic Survey, series of observations were made on effect of vibration on dam as check on calculated values and to determine whether 2 parts of dam act together in resisting vibrational forces. Assuming rigid foundation, computed value is about 0.14 sec. Observed period under vibration set up by wind alone, when water level was 110′ below crest, was 0.16–0.17 sec. Records obtained at time of heavy blast in quarry some 3 mi. distant indicated periods of from 0.14 to 0.18 sec., with dominant period of 0.16–0.17 sec. Shaking machine, essentially a set of motor-driven eccentric flywheels, produced 2 pronounced resonance peaks, one at 0.17 and other at 0.20 sec.: water at this time was 87′ below crest. No adequate explanation has been found for fact that there were 2 periods, but it was definitely concluded that shorter of 2, 0.17 sec., is natural period of dam. Tests showed conclusively that contact of concrete blocks on opposite sides of joint was sufficient to transmit vibration across it and into concrete beyond: in many cases, amplitude of vibration was larger on side of joint away from shaking machine. Significant observation is that, in this structure at least, amplitude of vibration is proportional to vibrating force. Tests were interpreted as verifying computed periods of vibration of dam and strengthened confidence of builders in its ability to resist earthquake shocks.—*R. E. Thompson.*

The Underlying Principles of Siphon Spillways. F. Johnstone Taylor. Contract Jour. (Br.) **119**: 356 and 431 (Aug. 3 and 10, '38). Siphon spillways may prove of value for reservoirs in valleys with flat slopes, in releasing water from canals where a rise of even 6″ may require costly, flood-retaining banks, and in minimizing the volume of a dam because this increases, roughly, as the cube of its height. Existence of the hollow spaces required in a masonry dam by the construction of a siphon has but little effect on pressure across base of the dam. Because of peculiar local conditions a siphon spillway was found particularly suited for the Laggan dam, resulting in max. storage with min.

flood crest. This type of siphon will prime itself under a head above the crest of $d/3$, where d is the depth of the siphon at the throat. The larger the crest radius the greater the permissible discharge but the larger the required priming head. While it is obviously possible to increase the vacuum in a siphon, too high a vacuum has been found to be a cause of vibration and cavitation. It might be suggested that in place of forming the siphon in the body of the dam it could be carried on the down-stream face. Although practicable the scheme offers no advantage in a high-head siphon. In one dam 150′ high, the outlets were fixed at 60′ to 70′ below crest because of the problem of the direction and velocity of water discharged, which must be thrown clear of foot of dam. Although the siphon spillway was introduced as far back as 1866, general adoption was slow because of the priming devices then considered necessary. Priming now takes place automatically when depth of water over the lip of the throat exceeds one-third the depth of the throat. As level of water in reservoir falls and the breaking point of siphonic action approaches air may be sucked in gulps to set up undesirable vibrations. To avoid this in the siphon spillways at the Laggan Dam, c. i. pipes were inserted at the top of the siphon together with specially designed valves. Siphons employed at Dunalstair intake dam of the Tummel development of the Grampians hydro-electric scheme have combined capacity of 2,550 c.f.s. Comparison is made between these and other siphons and particularly with the siphons in the O'Shaugnessy dam of the San Francisco water works.—*H. E. Babbitt.*

Construction Methods at Cajalco Reservoir. R. B. WARD. Civ. Eng. 8: 377 (Jun. '38) Cajalco reservoir, main storage unit for the distribution system of water from the Colorado River aqueduct, is located about 60 mi. east of Los Angeles. Initial and present development for storage capacity of 100,000 acre-ft., but ultimately may be extended to 225,000 acre-ft., requiring raising both dam and dike 49′. Principal structures involved include: diversion tunnel, outlet tunnel, two earth embankments, spillway, and outlet tower. The diversion tunnel is 9′ dia., 2,000′ long, and concrete-lined. The outlet tunnel is 14′ dia., 2,348′ long, and lined with welded steel backed with concrete. The steel is lined with reinforced concrete. Embankments are earth-fill protected by an 8″ reinforced concrete slab on the upstream slope. On the downstream slope there is a rock blanket 2′ thick with tile drains at toe. Dam is 210 high and 2,170′ long at crest. Dike is 94′ high and 7,575′ long at crest. Respective earth-fill yardages are 3,175,000 and 3,857,00. Construction methods for dam and dike were quite different, so far as foundation was concerned, because of differences in site and foundation conditions. After vegetation had been disposed of from the dike site, top soil was excavated leaving about 10″ of earth on top of suitable foundation. This was compacted by rolling 16 times with a sheep's-foot tamper. At site of main dam the creek-bed portion of the foundation was excavated to bedrock. Embankments of both dam and dike were composed of materials from borrow pits within the reservoir area. Compacted, in general, by 30,000 lb. sheep's foot rollers. Definite control of moisture content was maintained so that max. settlement over a period of 5 mos. for the dam has been only 0.22′ and for a period of 8 mos. for the dike has been 0.08′. Excavation for cut-off walls carried to rock at all

places. Grout holes put down of 30′ to 200′ at intervals of 6′ into which 3,643 cu. yd. of grout was forced under pressures up to 100 lb. per sq. in. In most cases the grout consisted of 1 part cement, 0.1 to 0.2 part bentonite, mixed with 3 parts of water. The 8″ protecting concrete slab on upstream slope was constructed in strips 18′ 9″ wide. Provision for expansion was made only at the top of the embankment where an asphalt-filled groove separated the slab from the parapet wall. Concrete cured under two coatings of coal tar pitch cut-back covered with 2 coatings of white wash. Spillway is uncontrolled overflow type with curved lip, 200 long and will pass 15,000 sec. ft. Releases from the reservoir will be controlled in a circular outlet tower 20′ dia. and 145′ high.—*H. E. Babbitt.*

Leakage Stopped with Grout Wall. ANON. Eng. News-Rec. **121**: 289 (Sep. 8 '38). Leakage through shale and sandstone abutment of north end of Alum Fork Dam, which impounds new water supply for Little Rock, Ark., developed during initial filling of reservoir. Pressure grouting reduced leakage from 360 to 35 g.p.m.; remaining leakage comes through solid rock formation 15′ below base of dam and is not believed to be dangerous. Grouting procedure is described. Diamond point core drills used for sinking 2½″ holes into rock and cement was selected as grouting material. Many of holes were tested with water applied under pressure prior to grouting. Latter was started with expanding stop set as low as water test indicated to be necessary and as each zone was grouted to refusal stop was raised, and finally casing was capped. Thin grout, 12 gal. water to 1 sack of cement, was used at first and slowly thickened to 5 gal. of water to each sack. Pressure specified not to exceed 100 lb. per sq. in. Occasionally, cinders and sawdust had to be introduced directly into hole in order to hold grout in large cracks. Grouting of 1 hole reduced leakage 54 g.p.m. Use of fluorescein found advantageous in locating and indicating velocity of flow. Total of 3,400′ was drilled, of which 1,940′ was overburden and 1,460′ rock: 4,300 bags of cement used. Total cost $16,800 or $51.60 per gal. per min. reduction in leakage. Leakage stopped more effectively and at less cost than would have been possible before or during construction of dam.—*R. E. Thompson.*

Compacting Cohesionless Material. RICHARDS M. STROHL. Eng. News-Rec. **120**: 850 (Jun. 16, '38). Recent studies of cohesionless materials have indicated that relatively high degree of stability can be gained by compacting approx. to critical density, i.e., density at which material will not contract when subjected to shearing deformation. In development of embankment section for rolled-fill earth dam at Franklin Falls on Pemigewasset R. in N. H. for flood control on Merrimack R., high cost of suitable core material, which must be imported from distance of 4½ mi., necessitated using as small thickness of core as would suffice to insure low seepage. Laboratory tests of critical densities of materials available and field tests of compaction were made and it was found that heavy crawler tractor followed by either sheepsfoot or disk roller making 6–8 passes over the materials in 12″ layers would produce satisfactory compaction at optimum moisture content of 22% by wt., i.e., just under saturation point. Sufficient no. of tests have not been

made to establish relation between void ratio and critical density. Latter varies with max. attainable density, grading and possibly with size and shape of grains. Analysis of stability of slopes should give consideration to critical density, as angle of internal friction varies with density of material.—*R. E. Thompson.*

Relations Between the Angle of Slope for Soils and Their Physical and Chemical Properties. RICHARD KOHLER. Jahrb. preuss. geol. Landesanstalt **57**: I: 475 ('37). Expts are described with object of ascertaining what, if any, relation exists between the safe angle of slope of a soil (e.g., in a dam or cutting) and its nature. Shearing strength, plasticity, particle size and effect of Na ion were detd. Results agreed with expectations but showed that no exact numerical relations between these properties and slipperiness are obtainable. The effect of the Na ion, while undoubtedly unfavorable to stability, may be largely masked. It is specially necessary to consider also the nature of the stratification, slope of the strata, existence and position of springs, and presence of clefts or cleavages.—*R. E. Thompson.*

Why Marshall Dam Failed. ANON. Eng. News-Rec. **120**:431 (Mar. 24, '38). Data from report of board of engrs. appointed to investigate cause of failure of Marshall Creek Dam in Kansas, gives somewhat exact details of foundation soil conditions to which failure is attributed. Failure was a downstream slump of a moonshaped medial section about 700′ long. Dam at time of failure had not been completed and only a small volume of water was stored in reservoir. No evidence of faulting was apparent in rock foundation. Exploratory holes were made over embankment and in valley for distance of 700′ upstream from dam and 3 representative sections were selected for detailed study. Foundation soils contain high percentage of clayey soils and many of them were found to contain almost 50% voids after having been subjected to gradually increasing embankment load for period of 12–18 mo. Failure resulted from plastic movement of foundation soils deficient in shearing strength in region about 100′ downstream from center line. Plastic movement was caused by overloading of foundation by combination of height of dam and steepness of slopes. Slice method of stability analysis was employed. Apparent factor of safety at failure was 0.84. However, actual factor of safety must have been only slightly under 1.00.—*R. E. Thompson.*

AQUEDUCTS

Delaware Water Supply System of the City of New York. ROGER W. ARMSTRONG. Civ. Eng. **8**: 581 (Sep. '38). As early as '21 the need for additional water supply was apparent and, after exhaustive investigation by the Board of Water Supply, it was apparent that Delaware R. plan would be most suitable. Estimated cost $278,000,000. Interstate rights to the water were settled by decree of the U. S. Supreme Court which allowed N. Y. City to divert up to 440 m.g.d. and provided for discharge of certain amounts of water into the Delaware R. in event of low water. Financial difficulties postponed start of construction to '36. Project proposed diversion of 370 m.g.d. from East Branch of Delaware R. and 70 m.g.d. from Neversink R., but volumes of

storage proposed will develop yields in excess of these quantities sufficient to supply water required for releases during dry flow. Considerable portion of watershed lies in Catskill State Park. All water collected will pass through Rondout reservoir, the aqueduct from which will be pressure tunnel connecting with existing system of the city. Chlorinating plants will be installed in outlet works of Rondout, West Branch, and Kensico reservoirs; a coagulating plant and an aerator will be at Kensico reservoir, together with connections to possible future filtration plant. The three dams will be earth embankment with concrete cutoff walls extending into rock. The pressure tunnel from Rondout reservoir will consist of 3 principal sections: (1) Rondout-West Branch, 45 mi. long, 13½′ dia., (2) West Branch-Kensico, 22 mi. long, 15′ dia., (3) Kensico-Hill View tunnel, 14 mi. long and 19½′ dia. All tunnels will be in deep rock, circular, and concrete lined. To speed construction contracts were let for deepest and most widely spaced shafts, together with those for the Kensico-Hill View tunnel, while other contracts were still in preparation. Precautions taken to prevent pollution of streams during progress of work. Shafts sunk by derricks for first 100′ where head frames were set having finished dia. of 14′. Progress thereafter about 100′ per month. Concrete linings placed as excavation of shafts advanced. In construction of Lackawack Dam caissons first sunk to investigate subsurface conditions to demonstrate feasibility of constructing cut-off wall by this method. Expected that within next few months contracts will be let for construction of Lackawack Dam, for completion of Rondout-West Branch tunnel, and for short section at north end of West Branch-Kensico tunnel. These, with contracts already in force, will provide for construction of all main structures between Rondout and Hill View reservoirs.—*H. E. Babbitt.*

Large Delivery Lines for the Colorado River Aqueduct System. Julian Hinds. Civ. Eng. 8: 469 (Jul. '38). Main Colorado R. aqueduct terminates at the Cajalco Reservoir about 60 mi. SE. of Los Angeles. Water will thence be conveyed by network of feeder lines to cities comprising the Metropolitan Water District. Designed av. flow of main aqueduct 1,500 c.f.s. Upper feeder from Cajalco to Eagle Rock has total length 62.5 mi., made up of 10.3 mi. steel pipe, 36 mi. concrete pipe, and 16.2 mi. tunnel, with designed capacity of 570 c.f.s. Steel used only for high-head siphon across the Santa Ana R. with max. head of 485′. Max. dia. of steel pipe 11′-6″, with max. thickness 31/32″. New welding methods and special pipe handling equipment were used. Steel pipe protected against corrosion by coal-tar enamel on inside and an outside gunite coating ¾″ thick. Except for few short monolithic canyon crossings, and necessary tunnels, all parts of the upper feeder, other than the Santa Ana R. siphon were precast concrete pipe with dia. up to 12′-8″, heads up to 292′, and thicknesses up to 13″. Concrete pipe manufactured in plants adjacent to line. After completion all pipes tested for leakage under full operating heads.—*H. E. Babbitt.*

New Boston Aqueduct for Quabbin Water. Anon. Eng. News-Rec. 121: 342 (Sep. 22, '38). Aided by PWA grant of $6,984,000, Metropolitan District Water Supply Commission of Boston will complete Quabbin Reservoir works

and provide additional aqueduct to carry Quabbin supply into city. Total cost of project will be $15,522,000. New aqueduct will permit cutting out of less desirable water from Sudbury Reservoir and holding it as emergency supply. About $11,000,000 will be required for aqueduct, extending 18½ mi. from Wachusett Aqueduct terminal to point near terminal of Weston Aqueduct: 3¾ mi. will be in rock tunnel and remainder in cut-and-cover pressure conduit of circular concrete section lined with steel. Work to be done at Quabbin Reservoir includes clearing 20,000 acres inside flowage line and building 2 low-flow regulating dams on Middle and Eastern Branches of Swift R. Filling scheduled to start next summer, after completion of Quabbin Dam.—*R. E. Thompson.*

Suspension Bridge for Water Pipe. Robert Sailer. Eng. News-Rec. **121**: 75 (Jul. 21, '38). In Ogden R. project of Bur. of Recl. 31″ steel siphon, crossing Ogden R. canyon, provides connection between South Ogden highline canal and Ogden conduit, which, running along northerly rim of canyon, brings water from reservoir formed by Pine View Dam. To carry steel siphon across canyon, unusual 360′ catenary suspension bridge was built, hanging 160′ above riverbed and anchored in walls of canyon. Estimates showed suspension bridge more economical and less exposed to falling rock than either siphon on bottom of canyon or steel arch bridge. Large deflections which occur in cable suspension bridges imposed many unusual requirements. Important that ample flexibility be provided in steel pipe and floor system for expansion, contraction and rotation. Special sleeve couplings with rubber gaskets gave necessary flexibility and allowed use of 20′ pipe sections. Each section supported midway between couplings by floorbeam. Main cables, 2 on each side, are 1¾″ with allowable working strength of 70,000 lb. per sq. in. Wind cables provide lateral rigidity. Siphon and bridge erected with aid of overhead cableway.—*R. E. Thompson.*

DISTRIBUTION—MAINS AND RESERVOIRS

Preparation of Elevated Water Supply Tanks for Winter Use. F. O. Malloy. Off. Bul., N. Dakota Water & Sew. Works Conf. **4**: 5: 8 (Nov. '37). Freezing of water in elevated tanks and risers is an important problem in the northwest. Aside from physical damage which may be caused there is the loss of use of tank capacity. Water storage space may be reduced over 50%; during past extreme winters this has amounted to 90%. Aluminum coated tanks average approximately one third more ice than a tank painted black. Careful attention should be paid to maintaining the expansion in proper working condition. The water storage space in the tank should be free from all obstruction and the overflow should be adequate to keep water level in the tank below the spider rods.—*P. H. E. A.*

Winter Protection of Water-Works Plants. Anon. Am. City. **52**: 11: 81 (Nov. '37). Following the severe winter of '35, the Amer. Water Works and Elec. Co. prepared cold weather instructions for the operators of its plants. The revised form of these is discussed here. Many plants need careful attention during cold weather and among them are structures extending into reser-

voirs, pump suctions with screens, pump housings, valves in unheated areas or positions, various sections of purification plants and the distr. system and meters. All of these are discussed and measures suggested for making them frost and cold proof so as to reduce maintenance and replacement costs.—*Arthur P. Miller.*

Supercooling and Freezing of Water. N. E. DORSEY. J. Research Nat. Bur. Standards **20**: 799 (Jun. '38). Preliminary report is given on a study of supercooling and freezing water. 37 specimens of water from various sources were shown to freeze at temperatures ranging from −3.3°C. (water from bottom of algae-covered pool) to −21.1°C. (vacuum distilled water from chromic solution no ebullition). Supercooling was shown not to be dependent upon extreme quiescence or upon the use of a minute volume, and each specimen had a definite reproducible "spontaneous-freezing-point" (S.F.P.). Samples from extreme layers of a column of distilled water in which sedimentation had been permitted to take place showed a definite difference in S.F.P. Observations show that within undetermined limits the S.F.P. is dependent on the size of the largest mote—the smaller the mote, the lower the S.F.P. Effect of agitation is briefly considered and three distinct types of freezing are recognized, namely (1) from the walls of container inward (sudden plunging into bath of temperature below S.F.P.), (2) spontaneous throughout container (slow cooling to S.F.P.), and (3) from single mote outward (slow cooling of specimen containing single well-washed bit of solid).—*T. E. Larson.*

Removing Incrustations from Water Pipes. DANTE I. CASALE. Bol. Obras Sanitarias Nacion (Buenos Aires) **2**: 498 (May '38). Article describes apparatus developed in the shops of Obras Sanitarias de la Nacion for cleaning incrusted water pipes. This apparatus consists of a small two story rectangular frame which supports on its upper part a 5 h.p. electric motor. Motor connected by chain drive to a horizontal shaft located on lower part of the frame. The end of the shaft is equipped with a clamp for attaching the cleaning tool. The frame is mounted on four wheels which ride on two guide rails. These rails are attached to the pipe to be cleaned in such manner as to keep the cleaning tool centered within the pipe. Cleaning tool consists of a rod at the end of which is attached a conically shaped scraper made of 6 triangular steel toothed knives radially located. For cleaning a long stretch of pipe, a trench is made and about 7′ pipe are cut out to make room for the cleaning tool and driving equipment. Cleaning tool is revolved by the motor while a hand operated worm and gear mechanism drives the carriage forward thus forcing the tool into the pipe. As tool advances, water is allowed to flow in opposite direction to wash out loosened material. When the driving mechanism reaches the pipe, the cleaning tool is disconnected, the carriage moved back, and an extension rod attached to the cleaning tool and the driving mechanism thus continuing the cleaning. Based on information obtained while cleaning a badly incrusted main, it is estimated that pipe in a city block can be cleaned in 8 hrs. with this equipment.—*J. M. Sanchis.*

Reclaiming and Cement Lining Old Cast Iron Pipes. WM. S. STAUB. W. W. and Sew. **85**: 117 (Feb. '38). A field-built set-up for cleaning and cement

lining salvaged cast iron pipe, together with details of procedure used, is described and illustrated. Final cost of reclaimed pipe ranged, depending on pipe size, from 18.5% to 34.7% of the cost of new pipe. Detailed tables of labor and costs are presented.—*H. E. Hudson, Jr.*

Maintenance and Repair Methods. NATHAN N. WOLPERT. W. W. Eng. **91**: 1132 (Aug. 31, '38). Description of Syracuse, N. Y. water dep't practices. Supplying pop. of 225,000 and area of 25.7 sq. mi., system contains 380 mi. of mains, 6667 valves and 5027 hydrants. Rather complete summary of operating practices given in 6 page illustrated article—*Martin Flentje.*

A Three-Story Reservoir of Novel Construction. ANON. Eng. News-Rec. **121**: 230 (Aug. 25, '38). Unique 10.5-mil. gal., 3-story reservoir recently completed in Nantes, France. Built of reinforced concrete, structure consists of thin arch sections supported by beams and buttresses. The 3 basins can be independently filled and emptied. Lowest basin is 243′ in outside diam., made up of series of 50 arches, each with span of 11.5′ and inclined at angle of 45°. Each arch can be deformed independently without danger to structure. Sides of middle basin, 216′ diam., are composed of thin vertical arches, and floor is formed by segmental arches extending beyond walls to form roof for lower basin. Top basin, patterned after others, is 179′ in diam. Tower in center houses 3 vertical wells containing valves for the 3 systems of distribution.—*R. E. Thompson.*

Blasting Hazards Reduced by Simple Safety Primer. ANON. Eng. Cont. Rec. **51**: 25: 25 (Jun. 22, '38). Brief illustrated description of primer developed for use on Colorado R. aqueduct. Av. of half stick of dynamite is saved per hole and there has not been single accidental primer explosion in 1,500,000 used on project to date. Essentially, device consists of wooden plug through which hole has been bored lengthwise and blasting cap inserted and glued in place. During handling and transportation, primers and dynamite are thus never in contact, and wooden covering provides safe protection when shovel strikes unexploded primer in muck pile. Directions are given for assembling safety primers and for loading drill holes.—*R. E. Thompson.*

HYDRAULICS—HYDRAULIC ENGINEERING

Water Hammer Pressures in Compound or Branched Pipes. ROBERT W. ANGUS. Proc. A.S.C.E. **64**: 133 (Jan. '38). Presents a brief introduction to the analytical method for the solution of water hammer problems in a long, straight pipe with valve or gate at lower end. General equations are given for the analytical solution of problems under such conditions. It becomes evident that much care must be exercised in application of the formulas, some of the calculations are long and involved, and there is possibility of error unless the computer is practiced in the method. Graphical methods proposed in the article are based on the general equations presented in the analytical discussion. The conditions under which water hammer and surge may occur, for which graphical methods of solution are presented, include: (1) A gate, completely closed, at the end of a long, straight pipe, (2) A gate, partially closed, at the end of a long straight pipe, (3) A gate at the end of a series of

pipes of different sizes connected end to end, (4) A gate at the end of a single straight pipe on which there is a surge tank, (5) A gate at the end of a long, straight pipe on which there is a dead-end branch, (6) A gate on the end of a long pipe line on which there is a loop, (7) A gate at the end of one branch of a pipe line from which water is being discharged into the atmosphere through another branch, (8) A turbine, penstock and draft tube, and (9) The breach of a water column occurring in a long discharge pipe line from a pump into a reservoir at a higher elevation. The breach may occur when the pump shuts down suddenly. In addition to presenting graphical methods for the solution of the above types of problems the effect of friction and velocity head are discussed. In general, it may be said that friction decreases the bad effects due to water hammer and computations made by neglecting it are usually on the safe side. The variety of the illustrations presented shows that the method is not only quick but is easily applied to problems met in practice. Method avoids tedious tracing of the various pressure waves, and study of the complicated reflection factors. Such experimental studies as have been made check the accuracy of the method. *Discussion. Ibid.* **64**: 1232 (Jun. '38) K. J. DEJUHASZ. For purpose of analyzing complicated inter-relationships, graphical method far superior to numerical computations. Examples given in this paper treat important practical cases and are highly instructive. Transient phenomena in elastic columns constitute a wide group, including, in addition to water hammer, spring surges, surges in gas columns, and electric transients, all having important applications in engineering. In dealing with fuel-injection phenomena writer has encountered difficulty in representing pipe friction in the diagram. Author's approach appears workable but open to objection that friction is assumed to be localized at definite points in the pipe, whereas actually it is distributed. HAROLD A. THOMAS. Since numerical methods of computation have certain advantages over graphical it seems desirable to present brief outline of the analytical methods for solving problems of type under consideration. In important cases each method should give valuable check on the other. Writer presents discussion of water hammer in frictionless, uniform pipe; the condition resulting from the discharge from a valve or gate into open air; and the general relation between head and discharge in frictionless pipe in which waves of pressure change are present. Illustrative problem is solved analytically and graphically and solutions shown to give identical results. *Ibid.* **64**: 1445 (Sep. '38). F. KNAPP. Approximate graphical methods have been devised to overcome lack of knowledge of the surge conditions affected by cavitation of the water column and friction losses distributed along the pipes. The elastic surge theory assumed the validity of Hooke's Law, i.e. stress varies as strain. The graphical method avoids the tedious tracing of the various pressure waves but it may lead to considerable errors, as shown in an example solved by the writer. The author's old-fashioned method of developing certain equations by differentiation has been superseded by simple physical considerations demonstrated by the writer in '37. MARTIN A. MASON, PIERRE F. DANEL, AND ANTOINE CRAYA. Among points of importance that have been omitted from consideration by the author may be included the case of a compound pipe consisting of a number of discontinuities provided by changes in section. The study of water-hammer conditions in compound pipes by the graphical method offers

a great advantage over the analytical method in that many of the factors that must be calculated in the analytical method are automatically considered in the graphical method. The method may be applied to any case involving the propagation of plane waves. A. A. KALINSKE. Prof. Angus' method is a graphical method for solving simultaneous equations and as such there is nothing mysterious about it. In general, the solution of any water-hammer problem involves the formulation of the relation between head and velocity at the control gate, at the end of a section of uniform conduit, at the other end of the same section an instant earlier, and the discharge relationships at the junction of pipe branches. Where friction is included the solution is greatly complicated and no satisfactory explanation of friction effect seems to exist for gradual gate closure. The writer believes that any further knowledge that is to be gained regarding water hammer will have to be obtained from experiments.—*H. E. Babbitt.*

Solution of Transmission Problems of a Water System. (*Discussion of previous paper*, see abstract J.A.W.W.A. **30**: 382 (Feb. '38).) Proc. A.S.C.E. **64**: 1202 (Jun. '38). F. KNAPP. Author's paper is less complicated than it first appears. If it had been made clear that four diagrams should have been drawn for the various flow demands solution would have been more easily understood. WESTON GAVETT. An advantage of author's method is use of natural coordinates for head and flow, permitting graphical summation of these values. Computation of the losses in a distribution system that may be checked by tests to an accuracy greater than the precision of the basic data should be sufficient for all except academic purposes.—*H. E. Babbitt.* (For additional discussion see also abstracts J.A.W.W.A. **30**: 707 (Apr. '38) and **30**: 1247 (Jul. '38)).

Coefficients of Discharge for Circular Section Gate Valves. PABLO BRISTAIN. Ingenieria (Mexico) **12**: 245 (Jul. '38). Valves used at canal intakes in the Delicias Irrigation District act as submerged orifices, flow through which is given by formula $Q = CA\sqrt{2gh}$. From this expression $C = \frac{Q}{A\sqrt{2gh}}$. The values of Q and h were determined in the usual manner. Author shows by means of trigonometric relations that the open area A when the disk of a circular section gate valve of diameter D is displaced vertically through a distance d can be computed by the equation $A = \frac{\pi r^2}{360}(\theta - \theta_1) + r^2 \sin \theta_1$ (where r is the radius of the gate valve) use being made of the further relations that $\cos \frac{\theta_1}{2} = \frac{d}{D}$ and that $\theta = 360 - \theta_1$. Determination of A for any value of d is facilitated by use of a graph plotted from a few calculated values of A using the corresponding ones of d as ordinates. Average coefficients found for 18" and 36" gate valves with values of d ranging from one-fifth to one-half the diameter, ranged from 0.62 to 0.71.—*J. M. Sanchis.*

A Compact Hydraulic Laboratory. WILLIAM MORTON. Civ. Eng. **8**: 611 (Sep. '38). Facilities for study of flood control problems have been provided at Univ. of Washington. Among the unusual features included are: a movable

observation bridge, a full-scale profilograph, a midget current-meter control panel, and an automatic sand feeder. These will be applied to models of the movable-bed type. Each model project will be operated as a unit in an outdoor flume 20′ x 80′. Laboratory water supply is by a 14″ x 12″ low-head pump, making 9 c.f.s. available. Connection to a larger motor makes 15 c.f.s. available.—*H. E. Babbitt.*

Report of the Research Committee for Years '35 –'36 and '36 –'37. Inst. of Civil Engineers (Br.) (Jan. '38). Report of more active participation in various research programs by this organization; comm. formed in '35 with 22 members. Increasing occurrence of severe corrosion of concrete in clay soils has led to comprehensive research (now being carried on) into effect of sulfate salts on concrete. Soil corrosion of pipes so far found to be more dependent upon nature of soil, its composition and physical condition than upon any variations in type of ferrous metal, or in the state of a non-ferrous metal. Study of vibratory methods of compacting concrete showed that acceleration imparted to mass of concrete of much greater importance than frequency of vibration as regards the ultimate strength of the concrete. If vibration sufficient to consolidate concrete, better surface appearance obtained with high-frequency vibration (up to 8000 vibrations per min.). Acceleration and duration of vibration necessary varies considerably with different concretes, little gain in strength however when acceleration increased from 5*g* to 10*g* or the time from 3 to 12 min. In studies made on velocity of flow formulas, Nikuradse experimenting with pipes in which roughnesses were of uniform size and closely spaced, found a comparatively abrupt transition from the "smooth" law (where roughness of surface has no effect on flow) at slow speeds to the "rough" law (where resistance depends upon surface texture) at high speeds, c.i., w.i., or galv. steel pipe give results indicating more gradual transition between 2 resistance laws. Resistance of pipe is function of size of elements of roughness and their distribution. Effect of age found to be as already published. Corrosion of water mains, incurring of electric shocks during repair of water mains, led to study of the grounding (earthing) to metal water pipes and mains. This grounding investigation being made co-operatively with other bodies, has just begun. Work being carried on in investigating earth-pressures and pile driving, wave-pressures, special cement for large dams, fish passes, bridge and steel structures also is reported on briefly.—*Martin E. Flentje.*

An Easily Constructed Orifice. Chester P. Baker and William A. McGrath. Ind. Eng. Chem.-Anal. Ed. **10:** 402 (Jul. '38). Instead of flanged castings of standard orifice, device described has ordinary union threaded throughout its length, with orifice plate held against shoulder formed by pipe in female side. Pipe ends may be machined or lead gasket used between pipe and orifice diaphragm. Manometer openings can be readily drilled and tapped after other parts are assembled. Orifice described is inexpensive and accurate, uses ordinary pipe fittings, and is easy to construct. Interior of orifice chamber is smooth, and orifice is easily removable and easy to center.—*Selma Gottlieb.*